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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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RELATION OF DEPTH TO CURVATURE OF CHANNELS

By H. C. Ripley,* M. Am. Soc. C. E.

Synopsis

The object of this paper is to record the results of some of the writer's investigations into the law of river hydraulics. As a result of these investigations, two empiric formulas have been devised by means of which the cross-profile of a channel may be computed. Some characteristics of the law were also disclosed, which are believed to be the most important discoveries in that law in more than fifty years. In conclusion, some practical applications of these formulas are given which furnish the solution to problems that heretofore have been considered not susceptible of solution.

Introduction

It is common knowledge with those who are familiar with river conditions, that a greater depth is found in bends of channels than in the straight reaches and that the elevation of the water surface on the concave side of the channel is higher than that on the convex side.

The writer first observed these conditions about 1872, when he was making a survey of the Red River in Louisiana for the removal of the Great Raft which obstructed about 100 miles of that stream. Since that time he has never ceased in his efforts to determine the cause of the greater depth in bends, and the relation of that depth to the curvature of the channel.

Many other engineers have also been interested in the same problem. By the use of a colored liquid, a French engineer found that in bends the water descended at the concave side of the channel, crossed over at the bottom, and came up on the convex side. An English engineer has demonstrated the helicoidal movement of water in bends, and another French Engineer, M. R. H. Gockinga, has devised the following formula (Formula (1)), for determining the difference in elevation of the water surface on opposite sides of the channel in bends:

$$Y = 0.235\ V^2 \log \left(1 - \frac{X}{R}\right) \qquad (1)$$

Note.—Written discussion on this paper will be closed with the April, 1926, *Proceedings*. When finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Cons. Engr., Detroit, Mich.

in which,

V = the longitudinal velocity, in meters per second;

R = the radius of curvature; and

X and Y = the co-ordinates of a point on the surface referred to a system of rectangular axes through the intersection of the axis of the channel and the water surface. That is, X equals one-half the width of the channel and Y equals one-half the difference in elevation on opposite sides of the channel.

For a width of 200 m., a radius of curvature of 500 m., and a depth sufficient to give a velocity of 1 m. per sec., with a longitudinal slope of 1 in 10 000, M. Gockinga found a cross-slope twice as great as the longitudinal slope.

The accuracy of Formula (1) has been confirmed by observations on the Mississippi River where differences of head of about 1 ft. have been noted on opposite banks of bends with mean mid-section longitudinal slopes of about 0.4 ft. per mile. These results seem to indicate conclusively that the increased depth in bends is caused by the helicoidal movement of the water induced by the centrifugal force of the current on the concave side of the channel.

The only attempts to determine the relation of depth to curvature known to the writer, were made by M. Fargue, a Frenchman, and the late Henry Mitchell, M. Am. Soc. C. E., of the United States Coast and Geodetic Survey. M. Fargue devised a formula for determining the radius of curvature necessary to produce a given maximum depth, which he designated as "Fargue's law of greatest depth". His formula is:

$$C = 0.03\,H^3 - 0.23\,H^2 + 0.78\,H - 0.76 \qquad (2)$$

in which,

C = the reciprocal of the radius of curvature, in kilometers; and

H = the low-water depth at the deepest point of the channel, in meters.

It is evident that Formula (2) is only applicable to the particular stream for which it was devised. Mr. Mitchell's formula is for determining the cross-profile of the Delaware River at Philadelphia, Pa., supposing it to be perfectly canalized so as to be reduced to the minimum width.*

His formula is:

$$Y = 33\,\frac{1\,100^2 - X^2}{1\,100^2} + \frac{289}{R}\left(\frac{1\,100^2 - X^2}{1\,100^2}\right) 2\,X \qquad (3)$$

in which,

R = the radius of curvature varying from 16 200 ft. to infinity; and

X and Y = the co-ordinates of the cross-profile, the origin being at the surface of the water in the center of the channel, and at a place where the width of the channel is 2 200 ft. and the depth 33 ft.

It will thus be seen that Formula (3) is quite limited in its application both as to curvature and character of channel.

* U. S. Coast Survey Report, 1878, p. 125.

THE WRITER'S FORMULAS

The writer's formulas are modifications of the Mitchell formula so as to make it of general application to all sizes of streams and to all degrees of curvature, and to every character of channel whether that channel occupies the entire waterway or only a part of it; or whether it is across an outer bar created artificially by the action of a single curved jetty or by straight or curved jetties in pairs.

These formulas are as follows:

$$Y = D\left(1 - \frac{X^2}{W^2}\right) + D\,\frac{17.52}{R}\left(1 - \frac{X^2}{W^2}\right)X \ldots\ldots\ldots\ldots (4)$$

$$Y = P\left(1 - \frac{X^2}{W^2}\right) + P\,\frac{26.28}{R}\left(1 - \frac{X^2}{W^2}\right)X \ldots\ldots\ldots\ldots (5)$$

in which,

D = the mean depth of the channel, in feet, multiplied by 1.445;
P = the mean depth of the channel, in feet, multiplied by 1.65;
W = the half width of the channel, in feet;
R = the radius of curvature, in feet, on the concave side of the channel; and
X and Y = the co-ordinates of the cross-profile of the channel, the origin being in the center of the channel at the surface of the water.

$$Y = D\left(1 - \frac{X^2}{W^2}\right) + D\,\frac{5.34}{R}\left(1 - \frac{X^2}{W^2}\right)X \ldots\ldots\ldots\ldots (6)$$

$$Y = P\left(1 - \frac{X^2}{W^2}\right) + P\,\frac{8.01}{R}\left(1 - \frac{X^2}{W^2}\right)X \ldots\ldots\ldots\ldots (7)$$

in which,

D = the mean depth of the channel, in meters, multiplied by 1.445;
P = the mean depth of the channel, in meters, multiplied by 1.65;
W = the half-width of the channel, in meters;
R = the radius of curvature, in meters, on the concave side of the channel; and
X and Y = the co-ordinates of the cross-profile of the channel, the origin being in the center of the channel at the surface of the water.

Notations.—

1.—Formula (4) is applicable to all streams of whatever size where the channel occupies the entire width of the waterway, with certain restrictions to be indicated subsequently.

2.—Formula (5) is applicable to channels not occupying the entire width of the waterway and to channels at entrances created by the action of a single curved jetty.

3.—Formula (6) is the same as Formula (4) adapted to measurements in meters.

4.—Formula (7) is the same as Formula (5) adapted to measurements in meters.

5.—The algebraic sign of X must be recognized in the solution of the formulas.

6.—Whenever the radius of curvature is less than 40 times the square root of the area of the channel, no further deepening of the channel results from the increased curvature. Hence, in such cases, the value to be given to R in the formula must be equal to $40 \sqrt{\text{area}}$.

7.—Whenever the radius of curvature is greater than about 50 times the square root of the area, the shape of the cross-profile does not conform strictly to that due to curvature. This result is presumably on account of the small helicoidal action when the curvature is slight, allowing other influences to predominate. It is believed, however, that where there are no other influences, the formula will almost exactly reproduce the actual cross-profile as shown in Fig. 5 (A), although the radius of curvature is 54.65 times the square root of the area $(54.65 \sqrt{\text{area}})$.

8.—Formulas (5) and (7) are not applicable to straight channels nor to curved ones where the radius of curvature exceeds about 110 times the square root of the area of the channel. (The exact figure has not been determined.)

9.—On a "cross-over bar", where the channel is neither on a curve, nor in a straight reach, the maximum depth is about 14½% less than its computed value.

10.—Formulas (5) and (7) give a width of channel at mean depth about 20% greater than the actual width, hence, the width of the channel at mean depth (a little more or less), as given by these formulas must be reduced by 20% to obtain its exact value.

Two important corollaries result from these findings: (1) That every artificial channel, whether it be a flume, an open aqueduct, a ditch, or a drainage canal, to discharge the greatest volume with a given slope, should have the form determined by Formula (4), both in straight and curved reaches; and (2) that bends in streams, the radii of curvature of which are greater than forty times the square root of the area of the cross-section, are constructive bends and tend to stability of channel both in position and depth; whereas those bends the radii of curvature of which are less than forty times the square root of the area of the cross-section are destructive bends, tending to cause a shifting of the position of the channel and to form cut-offs and crevasses.

Test of the Formulas

In order to test the accuracy of the results from the use of these formulas and to show their general application to all sizes of streams and to all degrees of curvature, the writer has computed a large number of cross-profiles of channels ranging in size from the Brazos River with a cross-sectional area of 5 000 sq. ft., a width of 336 ft., and a radius of curvature of 1 250 ft., to the Mississippi River with a cross-sectional area of 153 000 sq. ft., a width of 3 000 ft., and a radius of curvature of 36 600 ft.

These tests embrace the following streams: (1) Brazos River, Texas; (2) Mississippi River; (3) Aransas Pass, Texas (jetty channel); (4) Rio Grande do Sul, Brazil (Canal Norte); and (5) Southwest Pass, Mississippi River.

A comparison of the results of these computations with data on the actual channels is given in Tables 1 to 25, inclusive, and on Figs. 1 to 8, inclusive. In addition to these twenty-five tables and eight diagrams, Tables 26 to 29, inclusive, and Figs. 9 and 10 give some theoretical results which will be explained subsequently.

TABLE 1.—Brazos River (Fig. 1 (*A*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y.*	*Y.*	
255	0.00	0.00	Location, 3½ miles from the mouth.
250	0.60	0.60	Area of section, 5 293 sq. ft.
225	3.30	5.00	Width, 510 ft.
200	5.80	7.40	Mean depth, 10.38 ft.
175	7.90	9.60	Maximum depth, 15 ft.
150	9.80	11.60	Radius of curvature, infinite.
125	11.40	11.60	Value of D in Formula (4) = 15.
100	12.70	12.50	Area of the computed section is 3.9% less than that of the actual section.
75	13.70	13.00	
50	14.40	13.80	The maximum depth of the computed section is the same as that of the actual section.
25	14.90	14 50	
0	15.00	15.00	Computed from Formula (4).
− 25	14.90	14.50	The computed cross-profiles are shown on all diagrams in broken lines, and the actual ones in solid lines.
− 50	14.40	14.00	
− 75	13.70	13.70	
−100	12.70	12.60	
−125	11.40	11.40	
−150	9.80	10.30	
−175	7.90	9.20	
−200	5.80	7.50	
−225	3.30	4.40	
−250	0.60	0.60	
−255	0.00	0.00	

TABLE 2.—Brazos River (Fig. 1 (*B*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y.*	*Y.*	
195	0.00	0.00	Computed from Formula (4).
175	7.99	10.00	Location, 4¾ miles from the mouth.
150	15.58	18.00	Area of section, 5 384 sq. ft.
125	20.63	21.00	Width, 390 ft.
100	23.58	23.50	Mean depth, 13.81 ft.
75	24.70	24.25	Maximum depth, 25 ft.
50	24.26	22.50	Radius of curvature, 2 900 ft.
25	22.31	20.50	Value of D in Formula (4) = 19.95.
0	19.95	17.50	Area of computed section is 4% less than that of the actual section.
− 25	16.93	15.50	
− 50	13.00	13.00	This is Cross-Section No. 12 of the Official Survey of 1878.
− 75	9.30	11.00	
−100	5.82	9.00	
−125	2.87	6.50	
−150	0.77	4.50	
−175	− 0.23	2.30	
−195	0.00	0.00	

TABLE 3.—BRAZOS RIVER (FIG. 1 (*C*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y.*	*Y.*	
190	0.00	0.00	Computed from Formula (4).
175	5.91	11.50	Location, 13 miles from the mouth.
150	13.59	20.00	Area of section, 4 896 sq. ft.
125	18.52	22.00	Width, 380 ft.
100	21.87	21.00	Mean depth, 12.88 ft.
75	23.08	20.00	Maximum depth, 22.00 ft.
50	22.74	20.00	Radius of curvature, 1 900 ft.
25	21.15	17.00	Value of R in Formula (4) is 2 800 ft. which is 40 times the
0	18.61	16.00	square root of the area; this being the minimum radius of
− 25	15.43	15.00	curvature that will produce increased depth of channel.
− 50	11.90	12.00	Value of D in Formula (4) = 18.61.
− 75	8.34	9.50	Area of computed section is 3% less than that of the actual
−100	5.03	7.00	section.
−125	2.30	5.00	This is Cross-Section No. 5 of the Official Survey of 1878.
−150	0.43	4.00	
−175	− 0.27	2.50	
−190	0.00	0.00	

TABLE 4.—BRAZOS RIVER (FIG. 1 (*D*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y.*	*Y.*	
168	0.00	0.00	Computed from Formula (4).
150	8.64	5.50	Location, 13½ miles from the mouth.
125	17.50	14.50	Area of section, 5 179 sq. ft.
100	23.13	19.60	Width, 336 ft.
75	25.97	23.60	Mean depth, 15.41 ft.
50	26.48	25.00	Maximum depth, 25 ft.
25	25.09	23.50	Radius of curvature, 1 250 ft.
0	22.27	21.50	Value of R in Formula (4) = 2 879 ft., which is 40 times the
− 25	18.47	19.50	square root of the area, which is the minimum radius of
− 50	14.12	18.60	curvature to produce increased depth.
− 75	9.69	16.30	Value of D in Formula (4) = 22.27.
−100	5.63	13.60	Area of computed section is 4% less than that of the actual
−125	2.38	10.80	section, and the maximum depth is 6% greater.
−150	0.40	6.90	
−168	0.00	0.00	

TABLE 5.—BRAZOS RIVER (FIG. 1 (*E*)).

CO-ORDINATES, IN FEET.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
162	0.00	0.00	Computed from Formula (4).
150	4.90	4.00	Location, 13¾ miles from the mouth.
125	12.93	10.00	Area of section, 44 075 sq. ft.
100	18.25	14.50	Width, 324 ft.
75	21.24	16.00	Mean depth, 13.60 ft.
50	22.23	17.50	Maximum depth, 20 ft.
25	21.59	19.50	Radius of curvature, 3 500 ft., which is 52½ $\sqrt{\text{area}}$. (See
0	19.65	20.00	Notation 7.)
−25	16.79	19.00	Value of *R*, in Formula (4), = 3 500 ft.
−50	13.33	17.30	Value of *D*, in Formula (4), = 19.65 ft.
−75	9.64	15.90	Area of computed section is 4% less than that of the actual
−100	6.07	13.00	section and the computed maximum depth is 11.11% more
−125	2.97	9.00	than the actual depth.
−150	0.76	3.40	
−162	0.00	0.00	

TABLE 6.—BRAZOS RIVER (FIG. 2 (*A*)).

CO-ORDINATES, IN FEET.				General remarks.
Computed.			Actual.	
X.	*Y*.	*Y* + 5 ft.	*Y*.	
185	0.00	5.00	5.00	Computed from Formula (5). Location, 2 miles
175	4.72	9.72	14.00	from the mouth.
150	14.14	19.14	23.50	Total area of section, 375 ft. wide, 6 068 sq. ft.
125	20.30	25.30	30.00	Area of section, 5 ft. below water surface, 4 237
100	23.74	28.74	25.00	sq. ft.
75	25.32	30.32	21.00	Width, 5 ft. below water surface, 370 ft.
50	24.03	29.03	19.50	Mean depth, 5 ft. below water surface, 11.45 ft.
25	21.73	26.73	18.00	Maximum depth, 30 ft.
0	18.32	23.32	15.60	Value of *P* in Formula (5) = 18.32.
−25	14.25	19.25	14.50	Radius of curvature, 1 900 ft.
−50	9.93	14.93	13.60	Value of *R* in Formula (5) = 3 116 ft., which is 40
−75	6.24	11.24	13.50	times the square root of total area. (See Nota-
−100	2.20	7.20	13.50	tion 6.)
−125	−0.38	4.62	11.60	Area of computed section is 3.4 % more than that
−150	−1.20	3.80	8.60	of the actual section and the maximum depth
−175	−0.88	4.12	5.60	is 1% more. Channel occupies 65% of width of
−185	0.00	5.00	5.00	waterway.

TABLE 7.—Mississippi River (Fig. 3 (*A*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
1 450	0.00	0.00	Computed from Formula (4).
1 400	4.53	10.00	Location, 33° 58′ 30″ N. Lat.
1 200	21.08	32.00	Area of section, 134 275 sq. ft.
1 000	35.09	42.00	Width, 2 900 ft.
800	46.54	45.00	Mean depth, 46.30 ft.
600	55.36	47.00	Maximum depth, 66 ft.
400	61.81	53.00	Radius of curvature, infinity.
200	65.64	60.00	Value of *R* in Formula (4) = infinity.
0	66.91	64.00	Value of *D* in Formula (4) = 66.91.
— 200	65.64	65.00	Area of computed section is 5% less than that of the actual
— 400	61.81	59.00	section and the maximum depth is 2.9% more.
— 600	55.30	52.00	
— 800	46.54	50.00	
—1 000	35.09	47.00	
—1 200	21.08	40.00	
—1 400	4.53	15.00	
—1 450	0.00	0.00	

TABLE 8.—Mississippi River (Fig. 3 (*B*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
1 500	0.00	0.00	Computed from Formula (4).
1 400	15.85	17.00	Location, 33° 52′ 00″ N. Lat.
1 200	41.62	49.00	Area of section, 152 850 sq. ft.
1 000	60.48	82.00	Width, 3 000 ft.
800	72.85	81.00	Mean depth, 50.95 ft.
600	79.60	78.50	Maximum depth, 82 ft.
400	81.48	77.00	Radius of curvature, 36 600 ft.
200	79.23	75.00	Value of *R* in Formula (4) = 36 600 ft.
0	73.62	67.50	Value of *D* in Formula (4) = 73.62.
— 200	65.39	57.00	Area of computed section is 4% less than that of the actual
— 400	55.30	50.00	section and the maximum depth is 0.6% less.
— 600	44.08	43.00	
— 800	32.51	37.00	
—1 000	21.32	27.00	
—1 200	11.38	15.00	
—1 400	3.13	10.00	
—1 500	0.00	0.00	

TABLE 9.—Mississippi River (Fig. 4 (*A*)).

Co-Ordinates, in Feet.				General remarks.
Computed.			Actual.	
X.	*Y*.	*Y'*.	*Y*.	
1 120	0.00	0.00	0.00	Location, 30° 10′ 53″ N. lat.; 91° 08′ 45″ W. long.
1 000	107.71	53.09	18.00	Area of section, 206 670 sq. ft.
800	222.29	115.63	60.00	Width, 2 240 ft.
600	266.74	150.01	146.00	Mean depth, 92.26 ft.
400	256.45	161.14	172.00	Maximum depth, 178 ft.
200	216.84	153.94	150.00	Radius of curvature, 5 800 ft.
0	133.32	133.32	136.60	Column *Y* is computed for a value of *R* in Formula
− 200	51.30	104.20	122.00	(4) of 5 800 ft. shown in the diagram by dotted
− 400	−24.11	71.50	100.00	line.
− 600	−76.62	40.11	52.00	Column *Y'* is computed for a value of *R* in For-
− 800	−91.69	14.97	23.00	mula (4) of 18 184 ft., which is equal to 40 $\sqrt{\text{area}}$.
−1 000	−53.63	0.99	10.00	Value of *D* in Formula (4) = 133.32 ft.
−1 120	0.00	0.00	0.00	Area of computed section is 4.38% less than the
				actual section and the computed maximum depth
				is 6.4% less than the actual depth.

TABLE 10.—Mississippi River (Fig. 4 (*B*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
1 170	0.00	0.00	Location, 30° 00′ 07″ N. lat.; 90° 27′ 58″ W. long.
1 000	48.21	36.00	Area of section, 148 010 sq. ft.
800	85.94	88.00	Width, 2 340 ft.
600	106.06	108.00	Mean depth, 63.25 ft.
400	111.63	111.00	Maximum depth, 111 ft.
200	105.68	97.00	Radius of curvature, 18 300 ft., equal to 47.57 $\sqrt{\text{area}}$.
0	91.40	74.00	Value of *R* in Formula (4) = 18 300 ft.
− 200	71.70	65.00	Value of *D* in Formula (4) = 91.40 ft.
− 400	49.81	55.00	Area of computed section is 4.35% less than the actual area,
− 600	28.66	44.00	and the computed maximum depth is 0.57% more than the
− 800	11.40	38.00	actual depth.
−1 000	1.05	24.00	
−1 170	0.00	0.00	

TABLE 11.—Mississippi River (Fig. 4 (C)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	Y.	Y.	
1 493	0.00	0.00	Location, 30° 11′ 44″ N. lat.; 91° 01′ 40″ W. long.
1 400	21.96	60.00	Area of section, 191 669 sq. ft.
1 200	62.42	106.00	Width, 2 986 ft.
1 000	89.54	102.00	Mean depth, 64.18 ft.
800	105.86	96.00	Maximum depth, 106 ft.
600	112.81	91.00	Radius of curvature, 23 300 ft., equal to 53.3$\sqrt{\text{area}}$.
400	111.95	85.00	Value of R in Formula (4) = 23 300 ft.
200	104.77	80.00	Value of D in Formula (4) = 92.75 ft.
0	92.75	71.00	Area of the computed section is 4.04% less than the actual section, and the computed maximum depth is 6.42% more than the actual depth.
— 200	74.41	60.00	
— 400	60.22	52.00	
— 600	42.73	48.00	
— 800	26.38	48.00	
—1 000	12.74	43.00	
—1 200	3.24	28.00	
—1 400	—0.58	6.00	
—1 493	0.00	0.00	

TABLE 12.—Mississippi River (Fig. 5 (A)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	Y.	Y.	
1 165	0.00	0.00	Location, 29° 58′ 14″ N. lat.; 90° 15′ 35″ W. long.
1 000	41.01	58.00	Area of section, 133 932 sq. ft.
800	74.65	73.00	Width, 2 330 ft.
600	93.11	96.00	Mean depth, 57.48 ft.
400	98.94	92.00	Maximum depth, 96 ft.
200	94.73	90.00	Radius of curvature, 20 000 ft.
0	83.06	80.00	Value of R. in Formula (4) = 20 000 ft., equal to 54.65 $\sqrt{\text{area}}$.
— 200	66.49	68.00	Value of D. in Formula (4) = 83.06 ft.
— 400	47.60	47.00	Area of computed section is 4.35% less than the actual section, and the computed maximum depth is 3.06% more than the actual depth.
— 600	28.95	30.00	
— 800	13.13	26.00	
—1 000	2.71	20.00	
—1 165	0.00	0.00	

TABLE 13.—Mississippi River (Fig. 5 (*B*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
1 000	0.00	0.00	Location, 30° 03′ 04″ N. lat.; 90° 35′ 14″ W. long.
1 000	31.06	28.00	Area of section, 155 500 sq. ft.
800	77.06	114.00	Width, 2 200 ft.
600	104.12	104.00	Mean depth, 70.60 ft.
400	115.29	93.00	Maximum depth, 114 ft.
200	113.61	84.00	Radius of curvature, 23 300 ft.
0	102.14	82.00	Value of R in Formula (4) = 23 300 ft., equal to 59.09 $\sqrt{\text{area}}$.
− 200	83.91	68.00	Value of D in Formula (4) = 102.14 ft.
− 400	61.97	65.00	Area of computed section is 3.3% less than the actual section,
− 600	39.38	61.00	and the computed maximum depth is 1.75% more than the
− 800	19.18	56.00	actual depth.
−1 000	4.40	34.00	
−1 100	0.00	0.00	

TABLE 14.—Mississippi River (Fig. 5 (*C*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
1 240	0.00	0.00	Location, 30° 01′ 28″ N. lat.; 90° 42′ 20″ W. long.
1 200	10.00	22.00	Area of section, 165 720 sq. ft.
1 000	51.52	76.00	Width, 2 480 ft.
800	80.10	90.00	Mean depth, 66.82 ft.
600	97.29	80.00	Maximum depth, 90 ft.
400	104.72	76.00	Radius of curvature, 33 300 ft.
200	103.95	73.00	Value of R in Formula (4) = 33 300 ft. equal to 81.80 $\sqrt{\text{area}}$.
0	96.56	70.00	Value of D in Formula (4) = 96.56 ft.
−200	84.15	67.00	Area of computed section is 4.3% less than the actual section
−400	68.30	60.00	and the computed maximum depth is 16.36% more than the
−600	50.61	52.00	actual depth.
−800	32.64	52.00	
−1 000	16.00	50.00	
−1 200	2.26	14.00	
−1 240	0.00	0.00	

TABLE 15.—Aransas Pass (Fig. 2 (B)).

Co-Ordinates, in Feet.				General remarks.
Computed.			Actual.	
X.	Y.	$Y + 6$.	Y.	
250	0.00	6.00	6.00	Computed from Formula (5).
200	10.58	16.58	24.50	Location, jetty channel.
150	16.60	22.60	26.00	Total area of section, 510 ft. wide, 7 775 sq. ft.
100	18.91	24.91	23.30	Area of section, 6 ft. below water surface, 4 745 sq. ft.
50	18.32	24.32	20.00	Width, 6 ft. below water surface, 500 ft.
0	15.66	21.66	16.00	Mean depth, 6 ft. below water surface, 9.49 ft.
−50	11.74	17.74	12.70	Maximum depth, 26 ft.
−100	7.39	13.39	10.30	Radius of curvature, 6 000 ft.
−200	0.70	6.70	7.50	Value of P in Formula (5) = 15.66.
−250	0.00	6.00	6.00	Area of computed section is 5.4% more than that of the actual section and the maximum depth is 4.2% less.

TABLE 16.—Rio Grande do Sul, Brazil (Fig. 6 (A)).

Co-Ordinates, in Meters.			General remarks.
Computed.		Actual.	
X.	Y.	Y.	
388	0.00	0.00	Computed from Formula (6).
350	3.48	5.00	Location, Canal Norte, 2 800 m. above zero of jetties.
300	7.50	11.20	Area of section, 10 022 sq. m.
250	10.91	13.20	Width, 776 m.
200	13.70	14.00	Mean depth, 12 915 m.
150	15.87	14.40	Maximum depth, 18.4 m.
100	17.42	14.80	Radius of curvature = infinity.
50	18.35	15.60	Value of D, in Formula (6), = 18.66.
0	18.66	17.20	Value of W, in Formula (6), = 388 m.
−50	18.35	18.40	Area of computed section is 1.5% less than that of the actual section and the maximum depth is 1.4% more.
−100	17.42	18.40	Channel occupies the entire waterway.
−150	15.87	17.60	
−200	13.70	15.80	
−250	10.91	13.20	
−300	7.50	10.00	
−350	3.48	3.00	
−388	0.00	0.00	

TABLE 17.—Rio Grande do Sul, Brazil (Fig. 6 (*B*)).

Co-Ordinates, in Meters.				General remarks.
Computed.			Actual.	
X.	*Y.*	*Y* + 1	*Y.*	
690	0.00	1.00	1.00	Computed from Formula (7).
600	6.09	7.09	13.20	Location, 5 500 m. above the origin of the jetties.
500	10.82	11.82	15.90	Total area of section, 11 233 sq. m.
400	13.67	14.67	15.60	Area of section, 1 m. below water surface, 9 853 sq. m.
300	14.91	15.91	13.60	
200	14.82	15.82	12.00	Width, 1 m. below water surface, 1 380 m.
100	13.69	14.69	10.30	Mean depth of section, 1 m. below water surface, 7.14 m.
0	11.78	12.78	7.30	
−100	9.37	10.37	5.00	Maximum depth, 16 m.
−200	6.76	7.76	4.50	Radius of curvature, 4 285 m.
−300	4.19	5.19	4.45	Value of *P* in Formula (7) = 11.78.
−400	1.97	2.97	4.25	Area of computed section is 16% more than that of the actual section and the maximum depth is the same.
−500	0.36	1.36	3.00	
−600	−0.35	0.65	1.80	
−690	0.00	1.00	1.00	Channel occupies 52% of the width of the waterway.

TABLE 18.—Rio Grande do Sul, Brazil (Fig. 6 (*C*)).

Co-Ordinates, in Meters.				General remarks.
Computed.			Actual.	
X.	*Y.*	*Y* + 2.	*Y.*	
580	0.00	2.00	2.00	Computed from Formula (7).
500	4.62	6.62	10.30	Location, 6 500 m. above origin of jetties.
400	8.82	10.82	14.80	Total area of section, 11 109 sq. m.
300	11.58	13.58	15.60	Area of section, 2 m. below water surface, 8 769 sq. m.
200	12.92	14.92	14.80	
100	13.15	15.15	13.60	Width, 2 m. below water surface, 1 160 m.
0	12.46	14.46	11.50	Mean depth of section, 2 m. below water surface, 7.55 m.
−100	11.03	13.03	7.80	
−200	8.96	10.96	6.80	Maximum depth, 15.60 m. Value of *P* in Formula (7) = 12.46.
−300	6.73	8.73	6.60	
−400	4.24	6.24	5.70	Radius of curvature, 9 144 m.
−500	1.78	3.78	3.10	Area of computed section is 6.8% more than that of the actual section and the maximum depth is 2.9% less.
−580	0.00	2.00	2.00	
				Channel occupies 41% of the waterway.

TABLE 19.—Rio Grande do Sul, Brazil (Fig. 6 (*D*)).

Co-Ordinates, in Meters.				General remarks.
Computed.			Actual.	
X.	*Y*.	*Y* + 1.	*Y*.	
640	0.00	1.00	1.00	Computed from Formula (7.)
600	2.77	3.77	4.60	Location, 1 000 m. above Cocoruto.
500	8.18	9.18	11.30	Total area of section, 1 290 m. wide, 9 584 sq. m
400	11.48	12.48	14.80	Area of section, 1 m. below water surface,
300	13.11	14.11	16.40	sq. m.
200	13.18	14.18	14.50	Width, 1 m. below water surface, 1 280 m.
100	12.42	13.42	8.40	Mean depth, 1 m. below water surface, 6.48 m.
0	10.69	11.69	6.50	Maximum depth, 16.40 m.
−100	8.44	9.44	5.30	Radius of curvature, 4 200 m.
−200	5.97	6.97	4.60	Value of *P* in Formula (7) = 10.69.
−300	3.57	4.57	3.70	Area of computed section is 8.2% more than that
−400	1.54	2.54	3.10	of the actual section and the maximum depth
−500	0.19	1.19	2.40	is 13.4% less.
−600	−0.19	0.81	1.50	Channel occupies 52% of the width of the water-
−640	0.00	1.00	1.00	way.

TABLE 20.—Southwest Pass, Mississippi River (Fig. 7 (*A*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
600	0.00	0.00	Computed from Formula (4).
500	22.89	39.00	Location, 21 960 ft. from Head of Passes (Ockerson Section 48)
400	41.61	50.00	Area, 62 200 sq. ft.
300	56.18	56.50	Width, 1 200 ft.
200	66.58	58.50	Mean depth, 51.83 ft.
100	72.82	59.50	Maximum depth, 64 ft.
0	74.90	61.50	Value of *D* in Formula (4) = 74.90.
−100	72.82	62.30	Radius of curvature = infinity.
−200	66.58	62.50	Area of computed section is 4.3% less than that of the actual
−300	56.18	61.00	section and the maximum depth is 17% more.
−400	41.61	57.50	This section is on a "cross-over" bar (see Notation 9 *re* for-
−500	22.89	42.00	mulas for computing cross-profiles of channels).
−600	0.00	0.00	

TABLE 21.—Southwest Pass, Mississippi River (Fig. 7 (*B*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
710	0.00	0.00	Computed from Formula (4).
700	3.53	0.50	Location, 27 445 ft. from Head of Passes (Ockerson Section 59).
600	33.52	38.00	Area, 64 140 sq. ft.
500	54.73	72.00	Width, 1 420 ft.
400	68.20	71.50	Mean depth, 45.17 ft.
300	74.97	64.00	Maximum depth, 73 ft.
200	76.04	66.60	Value of D, in Formula (4), = 65.27.
100	72.47	60.00	Radius of curvature, 13 200 ft.
0	65.27	57.50	Area of computed section is 4% less than that of the actual section and the maximum depth is 4% more.
−100	55.49	53.30	
−200	44.14	49.00	
−300	32.27	42.00	
−400	20.90	36.80	
−500	11.07	31.00	
−600	3.80	22.00	
−700	0.13	0.70	
−710	0.00	0.00	

TABLE 22.—Southwest Pass, Mississippi River (Fig. 8 (*A*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
762.5	0.00	0.00	Computed from Formula (4).
700	21.30	20.00	Location, 37 600 ft. from Head of Passes (Ockerson Section 80).
600	47.91	80.00	Area, 67 280 sq. ft.
500	65.81	91.00	Width, 1 525 ft.
400	76.19	83.50	Mean depth, 44.12 ft.
300	80.10	77.00	Maximum depth, 91 ft.
200	78.65	67.50	Value of D in Formula (4) = 63.75.
100	72.81	56.50	Radius of curvature, 10 800 ft.
0	63.75	46.00	Area of computed section is 4.9% less than that of the actual section and the maximum depth is 12% less.
−100	52.49	38.60	
−200	40.07	32.60	
−300	27.66	27.00	
−400	10.23	22.50	
−500	6.87	18.60	
−600	0.65	16.00	
−700	− 1.46	10.50	
−762.5	0.00	0.00	

TABLE 23.—SOUTHWEST PASS, MISSISSIPPI RIVER (FIG. 8 (*B*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
600	0.00	0.00	Computed from Formula (4).
500	23.31	38.50	Location, 44 475 ft. from Head of Passes.
400	42.37	50.40	Area, 63 840 sq. ft.
300	57.20	56.50	Width, 1 200 ft.
200	67.80	63.50	Mean depth. 52.78 ft.
100	74.15	64.80	Maximum depth, 73 ft.
0	76.27	73.00	Radius of curvature = infinity.
—100	74.15	67.00	Value of *D* in Formula (4) = 76.27.
—200	67.80	65.50	Area of computed section is 4.33% less than that of the actual section and the maximum depth is 4.5% more.
—300	57.20	64.30	
—400	42.37	56.40	
—500	23.31	30.50	
—600	0.00	0.00	

TABLE 24.—SOUTHWEST PASS, MISSISSIPPI RIVER (FIG. 8 (*C*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
920	0.00	0.00	Computed from Formula (4).
900	3.86	10.00	Location, 53 605 ft. from Head of Passes (Ockerson Section 113).
800	20.88	33.00	Area, 67 540 sq. ft.
700	34.34	40.00	Width, 1 840 ft.
600	44.53	45.00	Mean depth, 36.71 ft.
500	51.72	51.50	Maximum depth, 55 ft.
400	56.23	55.00	Value of *D* in Formula (4) = 53.04.
300	58.33	54.60	Radius of curvature, 22 800 ft.
200	58.30	50.00	Area of computed section is 4% less than that of the actual section and the maximum depth is 6.1% more.
100	56.44	47.40	
0	53.04	42.00	
—100	48.38	45.50	
—200	42.76	42.60	
—300	36.47	43.00	
—400	29.79	36.50	
—500	23.00	33.30	
—600	16.43	30.70	
—700	10.32	27.00	
—800	4.98	20.00	
—900	0.70	5.00	
—920	0.00	0.00	

TABLE 25.—Southwest Pass, Mississippi River (Fig. 8 (*D*)).

Co-Ordinates, in Feet.			General remarks.
Computed.		Actual.	
X.	*Y*.	*Y*.	
1 395	0.00	0.00	Computed from Formula (4).
1 200	19.92	21.00	Location, 63 940 ft. from Head of Passes (Ockerson Section 134).
1 000	34.28	28.30	Area, 76 990 sq. ft.
800	43.21	40.00	Width, 2 790 ft.
600	47.47	46.00	Mean depth, 27.60 ft.
400	47.84	43.30	Maximum depth, 46 ft.
200	45.05	39.30	Value of *D* in Formula (4), = 39.87.
0	39.87	37.50	Radius of curvature, 22 800 ft.
− 200	33.05	34.50	Area of computed section is 4.2% less than that of the actual section and the maximum depth is 4% more.
− 400	25.34	30.30	
− 600	17.51	27.50	
− 800	10.31	23.60	
−1 000	4.48	20.50	
−1 200	0.80	11.50	
−1 395	0.00	0.00	

TABLE 26.—Rio Grande do Sul, Brazil (Fig. 9 (*A*)).

Co-Ordinates, in Meters.				General remarks.
Computed.				
X.	I.	II.	III.	
400	4.00	4.00	4.00	Computed from Formulas (6) and (7).
350	6.88	8.07	9.13	Location, Outer Bar (theoretical).
300	9.37	11.10	13.09	Assumed normal depth, 4 m.
250	11.48	13.48	15.97	Assumed area of cross-section, 10 000 sq. m.
200	13.21	15.18	17.89	Curve I is the cross-profile of the channel between parallel straight jetties, 800 m. apart.
150	14.55	16.24	18.95	
100	15.51	16.74	19.26	Curve II is the cross-profile of the channel between parallel curved jetties, 800 m. apart, with a radius of curvature of 5 000 m.
50	16.09	16.74	18.92	
0	16.28	16.74	18.02	
− 50	16.09	15.44	16.70	Curve III is the cross-profile of the channel for a single curved jetty with a radius of curvature of 5 000 m.
−100	15.51	14.28	15.04	
−150	14.55	12.86	13.15	
−200	13.21	11.24	11.15	Area of Curves I and II are each 2.8% less and the area of Curve III is 6.5% more than the assumed area of the channel.
−250	11.49	9.48	9.13	
−300	9.37	7.64	7.19	
−350	6.88	5.69	5.45	
−400	4.00	4.00	4.00	

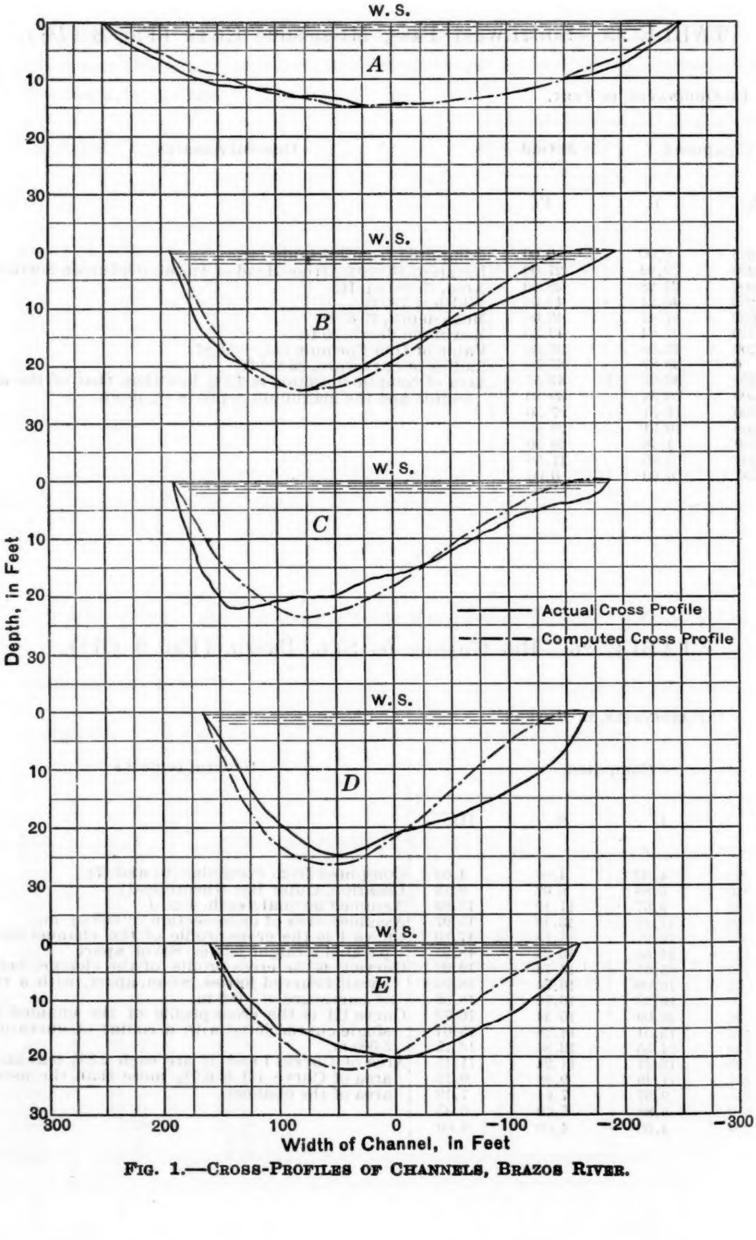


FIG. 1.—CROSS-PROFILES OF CHANNELS, BRAZOS RIVER.

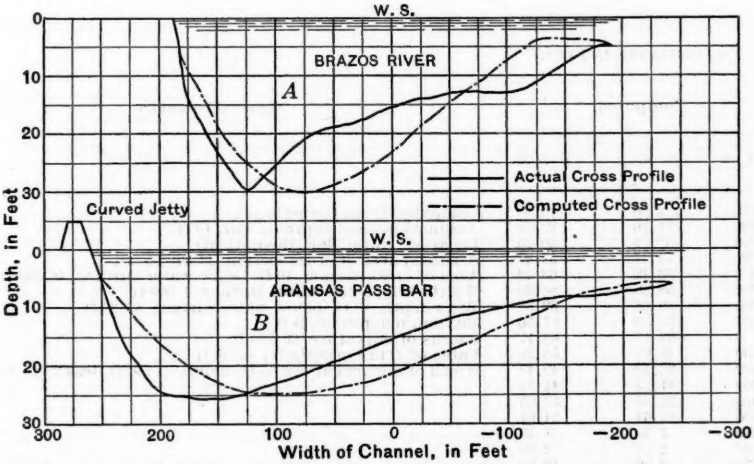


FIG. 2.—CROSS-PROFILES OF CHANNELS, BRAZOS RIVER AND ARANSAS PASS BAR.

TABLE 27.—SOUTHWEST PASS, MISSISSIPPI RIVER (FIG. 10 (A)).

CO-ORDINATES, IN FEET.				General remarks.
	Computed.			
X.	IV.	V.	VI.	
600	10.00	10.00	10.00	Computed from Formulas (4) and (5).
500	27.89	35.15	44.00	Location, Outer Bar (theoretical).
400	44.34	54.90	67.30	Assumed normal depth on Bar, 10 ft.
300	56.36	67.05	81.25	Assumed area of cross-section, 63 340 sq. ft.
200	64.95	73.40	87.22	Area of cross-section, 10 ft. below water surface, 51 340 sq. ft.
100	70.10	74.73	86.54	Assumed width of channel, 1 200 ft.
0	71.82	71.82	80.59	Curve IV is the cross-profile of channel between parallel straight
−100	70.10	65.47	70.72	jetties 1 200 ft. apart.
−200	64.95	56.50	58.28	Curve V is the cross-profile of channel between parallel curved
−300	56.36	45.67	44.63	jetties, 1 200 ft. apart, with a radius of curvature 22 800 ft.
−400	44.34	33.78	31.14	Curve VI is the cross-profile of channel for a single curved jetty
−500	27.89	20.63	19.14	with a radius of curvature of 22 800 ft.
−600	10.00	10.00	10.00	Areas of Curves IV and V are each 3.8% less than the assumed area of the section and the area of Curve VI is 7.4% more.

TABLE 28.—SOUTHWEST PASS, MISSISSIPPI RIVER (FIG. 10 (*B*)).

Co-Ordinates, in Feet. Computed. X.	Y.	Y + 10.	General remarks.
750	0.00	10.00	Computed from Formula (5).
700	11.70	21.70	Assumed normal depth on Bar, 10 ft.
600	30.75	40.75	Location, Outer Bar (theoretical).
500	44.47	54.47	Assumed area of cross-section, 63 340 sq. ft.
400	53.42	63.42	Area of cross-section, 10 ft. below water surface, 48 340 sq. ft.
300	58.20	68.20	Width, 10 ft. below water surface, 1 500 ft.
200	59.38	69.38	Mean depth, 10 ft. below water surface, 32.23 ft.
100	57.50	67.50	Maximum depth, 69.38 ft.
0	53.17	63.17	Radius of curvature, 26 000 ft.
—100	46.94	56.94	Value of P in Formula (5) = 53.17.
—200	39.40	49.40	Width of channel at the 35-ft. depth (1 005 ft. less 20%) = 804 ft.
—300	31.12	41.12	
—400	22.66	32.66	
—500	14.61	24.61	
—600	7.53	17.53	
—700	2.00	12.00	
—750	0.00	10.00	

TABLE 29.—RIO GRANDE DO SUL, BRAZIL (FIG. 9 (*B*)).

Co-Ordinates, in Meters. Computed. X.	Y.	Y + 4.	General remarks.
400	0.00	4.00	Computed from Formula (7).
350	4.42	8.42	Location, Outer Bar (theoretical).
300	7.77	11.77	Assumed normal depth on Bar, 4 m.
250	10.44	14.44	Assumed area of cross-section, 10 000 sq. m.
200	12.38	16.38	Area, 4 m. below water surface, 6 800 sq. m.
150	13.68	17.68	Width, 4 m. below water surface, 800 m.
100	14.38	18.38	Mean depth, 4 m. below water surface, 8.50 m.
50	14.41	18.41	Maximum depth, 18.41 m.
0	14.02	18.02	Radius of curvature, 9 000 m.
— 50	13.19	17.19	Value of P in Formula (7) = 14.02.
—100	11.97	15.97	Width of channel at the 10-m. depth (610 m. less 20%) = 488 m.
—150	10.44	14.44	
—200	8.64	12.64	
—250	6.64	10.64	
—300	4.49	8.49	
—350	2.14	6.14	
—400	0.00	4.00	

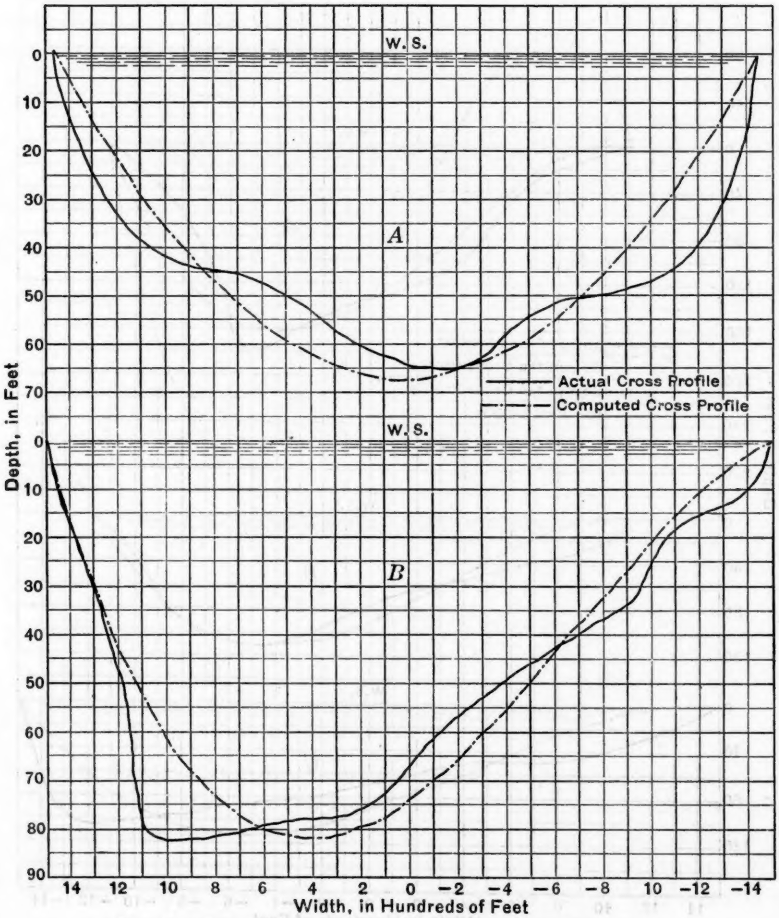


FIG. 3.—CROSS-PROFILES OF CHANNELS, MISSISSIPPI RIVER.

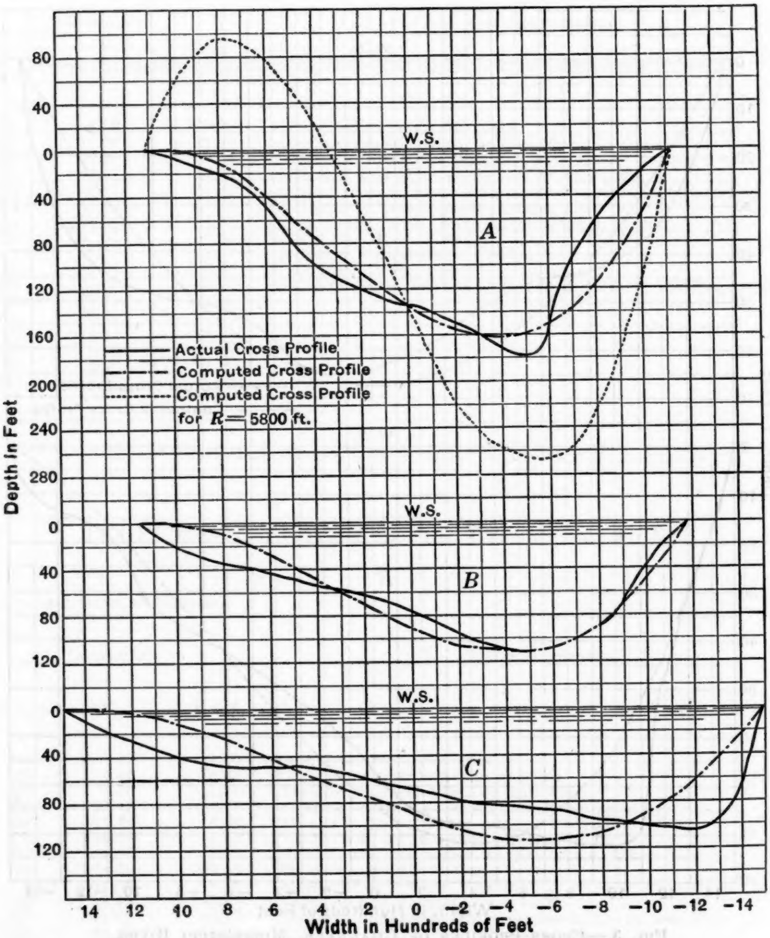


FIG. 4.—CROSS-PROFILES OF CHANNELS, MISSISSIPPI RIVER.

The explanatory remarks given in the tables furnish every essential detail necessary to a complete understanding of the results; so that further explanation would seem to be unnecessary. It should be stated, however, that in the use of Formulas (5) and (7), where the channel occupies only part of the waterway and, therefore, has only one bank, it is necessary to define the limit of the channel by the establishment of a second bank. This is accomplished by depressing the water surface until a depth is reached that will furnish a second bank and, at the same time, contain the entire actual channel. The co-ordinates of the cross-profile are then computed from this depressed water surface and the amount of depression afterward added, as shown in Tables 6, 15, 17, 18, 19, 28 and 29. These formulas have enabled the writer to dis-

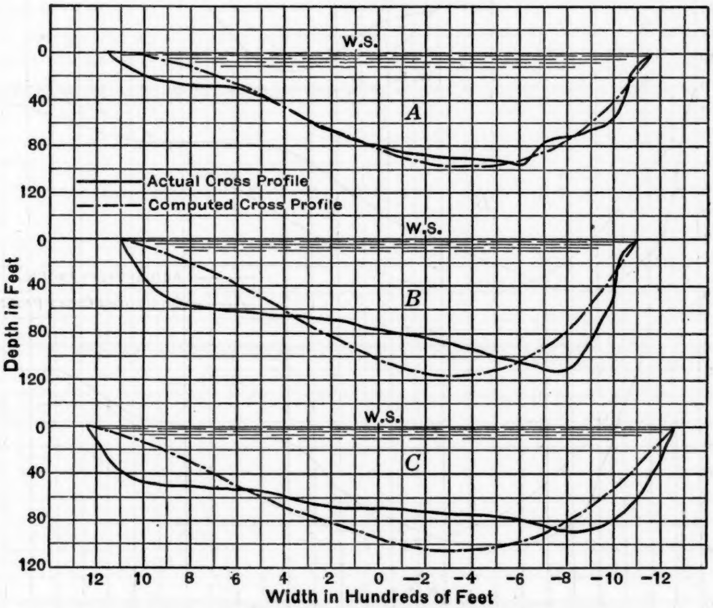


FIG. 5.—CROSS-PROFILES OF CHANNELS, MISSISSIPPI RIVER.

cover some characteristics in the law of river hydraulics not heretofore known, and it is believed they are the most important discoveries made in this law in more than fifty years. These discoveries may be enumerated as follows:

First.—The increasing curvature in bends increases the depth only to a certain point. This point is reached when the radius of curvature is equal to forty times the square root of the area of the cross-section $\left(40\sqrt{\text{area}}\right)$. This characteristic is illustrated in Tables 3, 4, and 9, and in Fig. 1 (*C*), Fig. 1 (*D*), and Fig. 4 (*A*).

Second.—Where the channel occupies the entire waterway, and the radius of curvature exceeds about fifty times the square root of the area of the cross-section $\left(50\sqrt{\text{area}}\right)$, the cross-profile may not conform strictly to that due to curvature; hence, the efficiency of the formula to reproduce results is restricted to those channels having a radius of curvature not exceeding this amount.

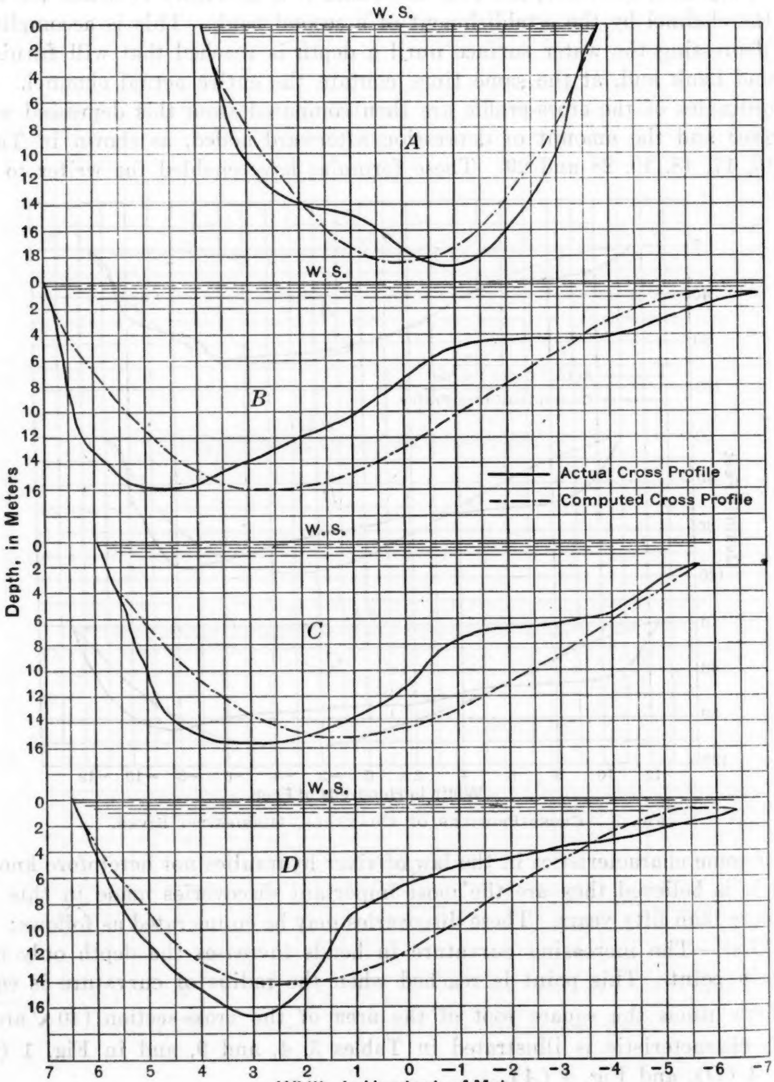


FIG. 6.—CROSS-PROFILES OF CHANNELS, RIO GRANDE DO SUL, BRAZIL.

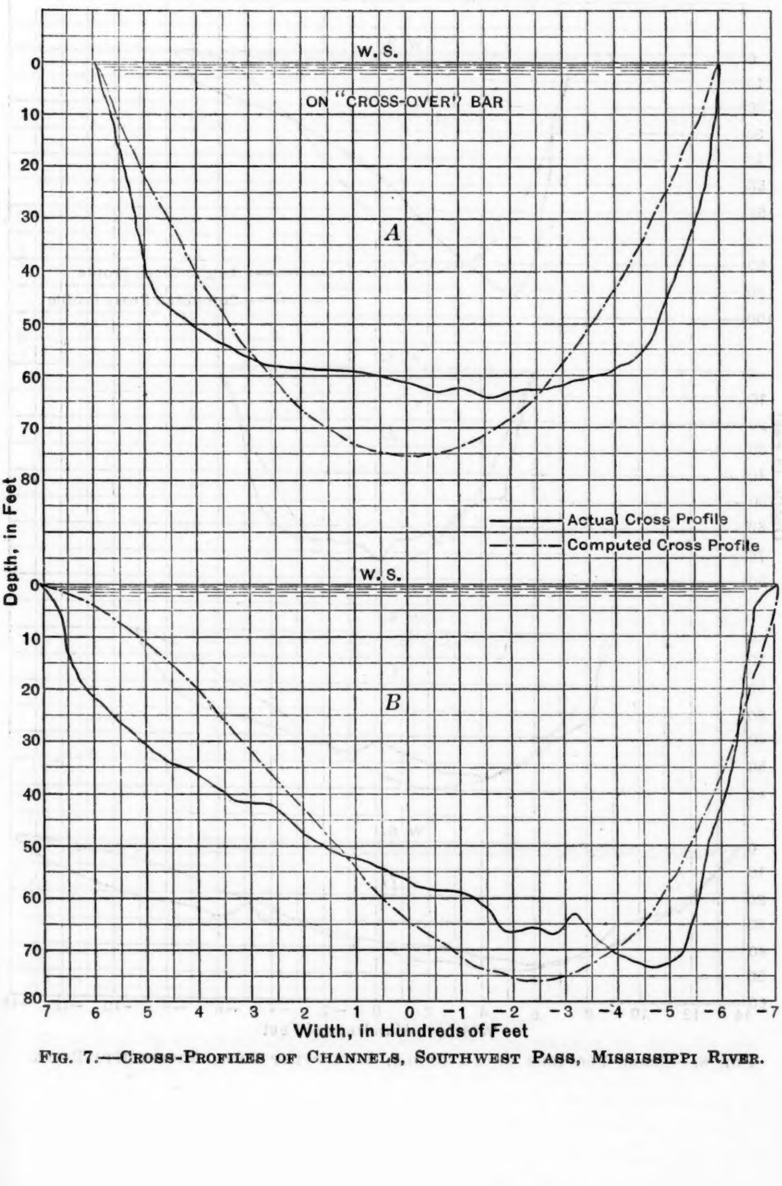


FIG. 7.—CROSS-PROFILES OF CHANNELS, SOUTHWEST PASS, MISSISSIPPI RIVER.

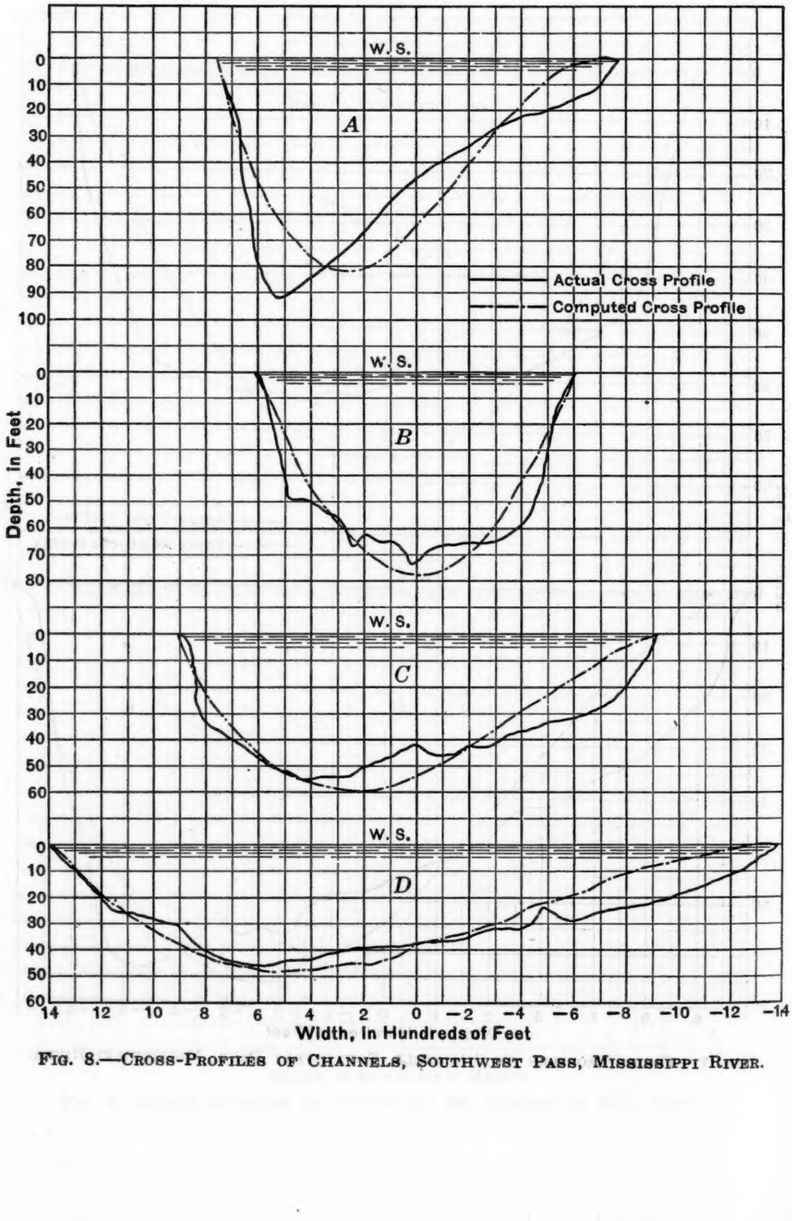


FIG. 8.—CROSS-PROFILES OF CHANNELS, SOUTHWEST PASS, MISSISSIPPI RIVER.

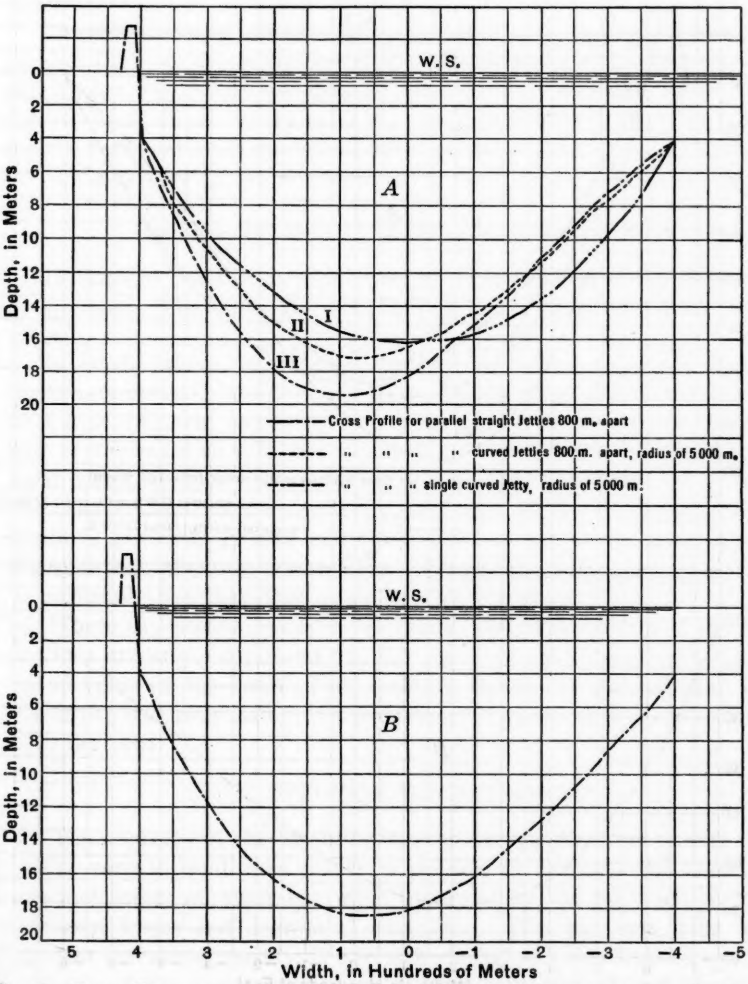


FIG. 9.—CROSS-PROFILES OF CHANNELS, OUTER BAR, RIO GRANDE DO SUL, BRAZIL.

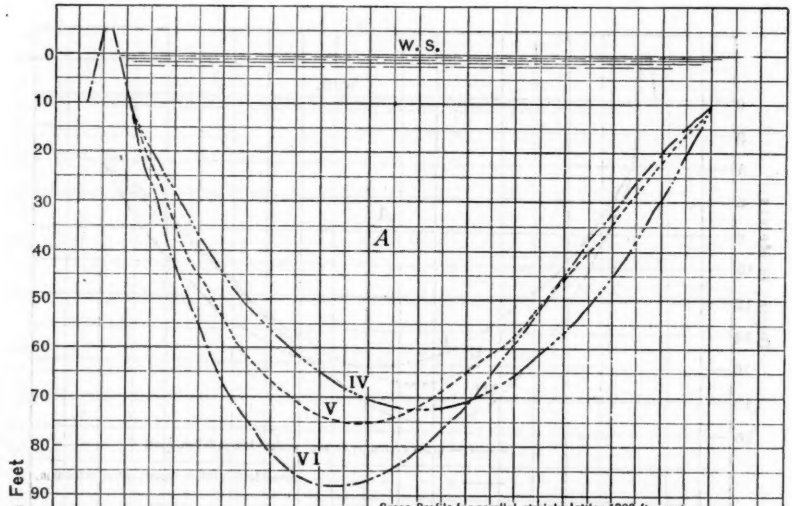


FIG. 10.—CROSS-PROFILES OF CHANNELS, SOUTHWEST PASS BAR, MISSISSIPPI RIVER.

These irregularities are illustrated in Table 5, Fig. 1 (E), Table 11, Fig. 4 (C), Table 12, Fig. 5 (A), Table 13, Fig. 5 (B), and Table 14, Fig. 5 (C). It appears, therefore, that in channels occupying the entire waterway, any curvature greater than that due to a radius of curvature equal to $40\sqrt{\text{area}}$ is ineffectual in producing increased depths; and any curvature less than that due to a radius of curvature equal to about $50\sqrt{\text{area}}$ may have irregularities which the formula cannot reproduce.

Third.—A single curved jetty will produce a deeper channel than two parallel jetties of the same curvature. This feature is illustrated in Tables 26 and 27 and Fig. 9 (A) and Fig. 10 (A). This is probably due to the greater facility with which the material excavated is dispersed and also may account for the fact that a single curved jetty will produce a channel across an outer bar without causing bar advance.

General Remarks on Results from Use of Formulas

In straight reaches the maximum depth is practically the same as its computed value. The width at mean depth is somewhat greater than its computed value.

In bends, where the channel occupies the entire waterway, the maximum depth is practically the same as its computed value with the single exception of Fig. 8 (A), but its position is generally somewhat closer to the concave side of the channel, with the single exception of Fig. 1 (D), and its width at mean depth is about the same or greater than its computed value. Where the channel does not occupy the entire width of the waterway, the maximum depth is generally greater and its position closer to the concave side of the channel than given by the formula. The width of the channel at mean depth is uniformly less than that given by the formula.

Where the channel occupies the entire waterway, the computed areas of all sections are uniformly about 4% less than the actual areas; whereas the computed value of the areas of all sections, where the channel does not occupy the entire waterway, are uniformly greater than the actual areas by an average of 8 per cent.

Practical Use of Formulas

The accuracy of the formulas in reproducing the cross-profiles of channels under various conditions makes it possible to compute the cross-profile of a channel across an outer bar when improved by the construction of jetties of different forms and degrees of curvature. This has been done for two important entrances, namely, Rio Grande do Sul and Southwest Pass, Mississippi River, and the results are given in Tables 26 and 27 and shown on Fig. 9 (A) and Fig. 10 (A). These computations are made for parallel straight jetties, parallel curved jetties, and a single curved jetty of the same curvature. The superiority of the single curved jetty thus becomes very apparent not only over the parallel straight jetties but over the parallel curved jetties as well.

Inasmuch as there is a great excess in the depth beyond that required for navigation, it would be better in either case to give to the jetty a greater

radius of curvature. This has been done and the results are given in Tables 28 and 29 and shown on Fig. 10 (*B*) and Fig. 9 (*B*), respectively.

For Rio Grande do Sul, the initial width is the same as in Fig. 1 (*A*), and the radius of curvature has a length of 9 000 m. This gives a channel with a maximum depth of 18.41 m. and a width of channel, at the 10-m. depth, of 610 m., which being reduced 20%, in accordance with Notation 10 (page 1910) will give an actual width of 488 m.

For Southwest Pass, the initial width is taken at 1 500 ft. and the radius of curvature at 26 000 ft. This gives a maximum depth of 69.38 ft., and a width of channel, at the 35-ft. depth, of 1 005 ft., which, being reduced by 20% in accordance with Notation 10, will give an actual width of 804 ft. This width is only 4% less than is found in the Pass itself in a stretch of 3 500 ft., from Sections 74 to 81, where the average width between the 35-ft. curves is 837 ft. and the minimum width is 765 ft., and the channel will be more nearly straight than in the Pass itself for much of its length.

In the use of these formulas, it may sometimes be desirable to determine the maximum depth without making repeated computations for that purpose. By making the differential coefficient of the formulas equal to zero and solving for X, a value is determined which, if substituted in the formula, gives the maximum value of Y at once. Thus, taking Formula (4):

$$Y = D\left(1 - \frac{X^2}{W^2}\right) + D\,\frac{17.52}{R}\left(1 - \frac{X^2}{W^2}\right) X$$

$$\frac{d\,Y}{d\,X} = -\,2\,D\,\frac{X}{W^2} + D\,\frac{17.52}{R} - 3\,D\,\frac{17.52}{R}\,\frac{X^2}{W^2} = 0$$

Letting $\frac{17.52}{R} = C$, and dividing by D:

$$\frac{2\,X}{W^2} + C - \frac{3\,C\,X^2}{W^2} = 0$$

which by simple transformation becomes:

$$X^2 + \frac{2\,X}{3\,C} = \frac{1}{3}\,W^2$$

whence,

$$X + \frac{1}{3\,C} = \pm\sqrt{\frac{W^2}{3} + \frac{1}{9\,C^2}}$$

and,

$$X = \pm\sqrt{\frac{W^2}{3} + \frac{1}{9\,C^2}} - \frac{1}{3\,C}$$

By restoring the value of C:

$$X = \pm\sqrt{\frac{W^2}{3} + \frac{1}{9}\left(\frac{R}{17.52}\right)^2} - \frac{1}{3}\,\frac{R}{17.52}$$

which becomes:

$$X = \pm\sqrt{\frac{W^2}{3} + \left(\frac{R}{52.56}\right)^2} - \frac{R}{52.56} \cdots\cdots\cdots\cdots (a)$$

In like manner, Equations (*b*), (*c*), and (*d*), are found, which, together with Equation (*a*), are, as follows:

$$X = \pm\sqrt{\frac{W^2}{3} + \left(\frac{R}{52.56}\right)^2} - \frac{R}{52.56} \ldots\ldots\ldots\ldots\ldots (a)$$

$$X = \pm\sqrt{\frac{W^2}{3} + \left(\frac{R}{78.84}\right)^2} - \frac{R}{78.84} \ldots\ldots\ldots\ldots\ldots (b)$$

$$X = \pm\sqrt{\frac{W^2}{3} + \left(\frac{R}{16.02}\right)^2} - \frac{R}{16.02} \ldots\ldots\ldots\ldots\ldots (c)$$

$$X = \pm\sqrt{\frac{W^2}{3} + \left(\frac{R}{24.03}\right)^2} - \frac{R}{24.03} \ldots\ldots\ldots\ldots\ldots (d)$$

Equations (*a*), (*b*), (*c*) and (*d*) are to be used with Formulas (4), (5), (6) and (7), respectively. The positive value of X will give the maximum value to Y in every case. A minimum value of X will generally not occur and, therefore, need not be considered.

Some important applications of Formulas (5) and (7) have already been given for determining the cross-profiles of channels on the outer bars of Southwest Pass, Mississippi River, and at Rio Grande do Sul, Brazil. Numerous other applications of these formulas will be found in the improvement of the channel in streams. For a cross-over bar where greater depth is desirable, Formula (4) furnishes a ready means for calculating the contraction necessary to give the required depth. In Formula (4), D represents the central depth which is 17% greater than the actual depth on a cross-over bar. (See Table 4.) To find the contraction necessary, reduce W in Formula (4) until the value of D, reduced by 17%, equals the depth required.

Analyzing Formula (4), it will be seen that D is the central depth, W is the half-width of the stream, and R is the radius of curvature. Hence, if the central depth, the width, and radius of curvature are known, it is possible to compute the mean depth, the maximum depth, and the area of the cross-section; for the mean depth is equal to D divided by 1.445, the area is equal to the mean depth multiplied by the width, and the maximum depth is found by substituting the value of X obtained from Equation (*a*) in Formula (4) and solving for Y.

Some interesting facts are thus determined in regard to the Amazon River which, at a point about 300 miles from its mouth, has a central depth of 360 ft., a width of 18 240 ft., and a radius of curvature of 273 600 ft. From these data the computation gives a mean depth of 249 ft., an area of 4 544 200 sq. ft., and a maximum depth of 387 ft.

The Mississippi River, as shown in Table 8, has a central depth of 67.5 ft., a width of 3 000 ft., a radius of curvature of 36 600 ft., a mean depth of 51 ft., an area of 152 850 sq. ft., and a maximum depth of 82 ft.

That is, the Amazon has 6 times the width, 30 times the area of cross-section, 5 times the mean depth and 4.7 times the maximum depth of the Mississippi River.

The relative discharge of these two great rivers is unknown, but some idea of it may be formed from the fact that the Amazon at flood shows river water

in the ocean 250 miles from its mouth, whereas the Mississippi River at flood shows river water in the Gulf of Mexico only 25 miles.

Conclusions

These formulas and the discoveries enumerated will be of the greatest value in the regularization of rivers and in the improvement of outer bar channels. They furnish a proof that the jetty channel at Aransas Pass, Texas, was caused by the action of the single curved jetty alone and that any effect produced by the straight jetty to the south of it has been detrimental. They also furnish the means by which the cross-profile of any proposed channel across an outer bar may be computed with remarkable accuracy, heretofore impossible of determination. They show that there is a limit to the effective curvature of a channel and that any attempt to exceed this limit will be fruitless.

Professor Mitchell has shown, in his investigations of the estuary of the Delaware River embracing a length of 52 miles from Philadelphia, Pa., down, "that the mean depth is the same in all sections and is equal to 18.64 ft. from all soundings for 46 nautical miles".*

Now, as the maximum depth in the straight reaches of any stream is equal to the mean depth multiplied by a constant, and the maximum depth in bends is equal to the mean depth multiplied by the same constant plus the effect due to curvature, it follows that the maximum depth in all straight reaches will be the same. Any attempt, therefore, to deepen the channel by dredging will disturb this persistent tendency to uniformity of depth. Hence, it would appear that it is not practicable to secure any permanent improvement in the navigable channel of that river by dredging. It is evident, therefore, that the logical method would be to convert the straight reaches into channels of suitable curvature by means of training walls and to let Nature do the rest. In this way equality of mean depth would be preserved and the increased depth secured would remain permanent forever.

*U. S. Coast and Geodetic Survey, Report, 1883, pp. 239-245.

NOTES ON SHEAR IN COMPRESSION MEMBERS

By Ralph E. Goodwin,* Assoc. M. Am. Soc. C. E.

Synopsis

The object of this paper is to apply the principles of Mechanics to the subject of shear in compression members. A method is presented by which given conditions can be analyzed and those which produce the greatest shear can be determined, thus enabling the engineer to design for the most dangerous conditions that he considers it advisable to assume.

The relationship between shear and the slope of the column axis herein given has already been published elsewhere, but as far as the writer is aware, the interesting possibilities of this relationship have not previously been developed.

In spite of the general discussion concerning shear in compression members, which took place following the collapse of the first Quebec Bridge in 1907, the fundamental principles governing shear are not so generally understood by engineers as could be desired, in view of the importance of the subject. On account of the difficulty of applying these principles directly to practical problems, their significance appears to have been overlooked by the Engineering Profession. As the slope of the column axis is a controlling condition, but is seldom known, a theoretical analysis might seem at first glance to be of no practical value. Such a conclusion, however, would be far from the truth. Closer study shows that it is impossible to make safe assumptions without a knowledge of the fundamental principles involved. Without such knowledge the engineer can not even know what possibilities exist, and, therefore, can not design for the worst possible condition. It will be shown that the actual shear in Member *A*-9-*L* of the first Quebec Bridge must have been approximately double that indicated by the beam analogy which is used by some of the standard engineering reference books and present-day specifications. This fact could not have been determined or explained without a knowledge of the fundamental principles involved in the problem.

It is not claimed that the analysis given in this paper reduces the computation of shear to a mathematical certainty, but it does furnish a scientific basis for the necessary assumptions, and leads to the following interesting conclusions:

Note.—Written discussion on this paper will be closed with the April, 1926, *Proceedings*. When finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Instr., Civ. Eng., Coll. of the City of New York, New York, N. Y.

1.—The minimum possible shear for a given deflection, instead of the maximum value, which is desired in designing, is given by the time-honored assumption that, in respect to shear, a column is similar to a simple beam with uniform load. If the maximum deflection occurs anywhere except at the middle of the column, the maximum shear will be greater than that indicated by the beam analogy.

2.—Purely theoretical deflection curves which show the maximum deflection occurring at the middle of the column, also gives the minimum possible shear for a given deflection.

The reason the actual shear in Member *A-9-L* of the first Quebec Bridge was twice that indicated by the beam analogy is that it deflected in an **S** curve (see Fig. 2), and, in general, an **S**-shaped deflection curve will produce double the shear indicated by the beam analogy or by the commonly assumed deflection curves.

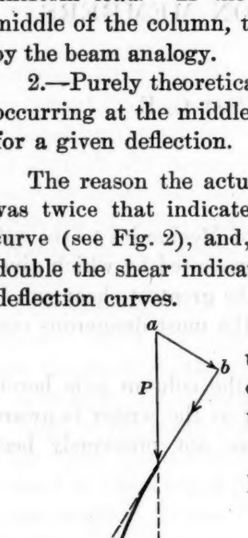


Fig. 1.—Column with Hinged Ends Deflected Under Load.

Fig. 1 shows a column with hinged ends deflected under load. The shear at the end of this column is obviously *ab*, the component of the load which is normal to the axis of the column, or

$$\text{Shear} = ab = P \sin G$$

For a parabola, $cd = f$, so that:

$$\tan G = \frac{2\,f}{\frac{L}{2}} = \frac{4\,f}{L}$$

As the sine of small angles is approximately equal to the tangent:

$$\sin G = \tan G \text{ (approximately)} = \frac{4\,f}{L} \text{ (approximately)}.$$

Therefore,

$$\text{Shear} = P \sin G = \frac{4\,P\,f}{L} \text{ (very nearly)}.$$

As Pf is the maximum bending moment:

$$\text{Shear} = \frac{4\,M}{L}$$

The illustration chosen is one in which the maximum shear occurs at the end of the column. In certain other cases the maximum shear will be elsewhere, but it will always be at a point where a tangent to the deflection curve makes the maximum angle with the direction of the applied load. This last statement may be checked from a different point of view. From the principles of Mechanics it is known that the greatest shear will occur at the point where the bending moment changes most rapidly. For a column, this will be where the tangent to the deflection curve makes the largest angle with the direction of the applied load, as stated previously.

In applying the equations given, it should be noted that the theoretical deflection curve shown in Fig. 1 produces the minimum possible shear. If the

same maximum deflection occurred at the quarter-point, the inclination of the axis at the end would be greatly increased, and the shear would be increased proportionately. If an **S**-shaped deflection curve were assumed, the shear would be double that produced by the curve of Fig. 1, or $\frac{8\,M}{L}$. The expression, shear $= P \sin G$, however, will always be true, if G is the angle between a tangent to the deflection curve at the point in question and the direction of the applied load.

A frequently used allowance for shear in columns is $0.02\,P$. Exactly what this means can be found by applying the principles set forth. It means that the maximum value of the angle, G, must not exceed $1°\,09'$, if the column is to be safe. For the theoretical case shown in Fig. 1, it also means that:

$$\text{Shear} = \frac{4\,P\,f}{L} \text{ must not exceed } 0.02\,P$$

and, therefore,

$$f \text{ must not exceed } \frac{0.02\,P\,L}{4\,P} = 0.005\,L$$

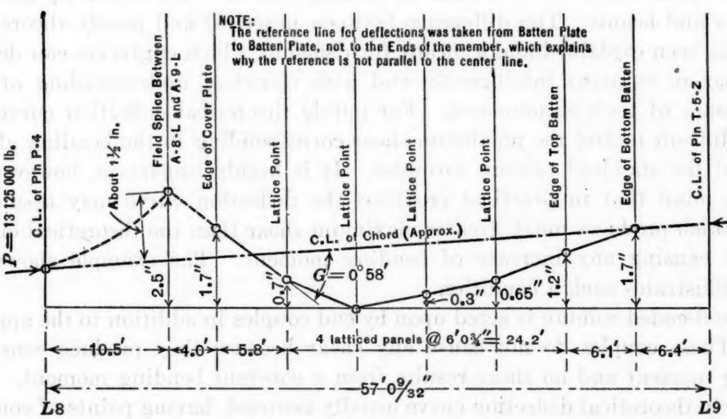


FIG. 2.—DEFLECTION OF CHORD MEMBER A-9-*L*, FIRST QUEBEC BRIDGE.

The application of these principles to the much discussed chord member, *A-9-L*, of the first Quebec Bridge, which collapsed, is interesting. The latticing of this member was double, at 45 degrees. There were two ⅞-in. rivets in the end of each lattice-bar. According to the report of the Royal Investigating Commission, the ultimate strength of the rivets in single shear was 50 960 lb. per sq. in., and the total compressive load, P, on the chord at the time of failure was 13 125 000 lb. The maximum total shear indicated by the expression, $P \sin G$, for conditions just prior to failure (Fig. 2), is 223 125 lb., computed as follows:

$$\sin G = 0.017 \text{ (found by scaling Fig. 2)}$$

$$P \sin G = 13\,125\,000 \times 0.017 = 223\,125 \text{ lb.}$$

The total shear indicated by the beam analogy for a maximum deflection, $f = 1\frac{1}{2}$ in. (the maximum deflection reported just before failure) and a length, $L = 57.02$ ft. $= 684.3$ in., is:

$$\text{Shear} = \frac{4\,P\,f}{L} = 115\,100 \text{ lb.}$$

The ultimate total shear necessary to cause failure of the latticing on this member, computed from the value in single shear of the end rivets in the lattice-bars, is:

$$8 \times 0.601 \times 50\,960 \times \sin 45^\circ = 173\,224 \text{ lb.}$$

The method here used for computing the shear corresponding to a given stress in the lattice-bars is the approximate one that assumes the total transverse shear to be resisted by the lattice-bars. The conclusions are not changed, however, when the shear is computed by the rigorous method; that is, the actual shear (as indicated by the deflection curve and the load) was greater than the shear theoretically necessary to cause failure, and was nearly twice as great as the shear indicated by the beam analogy.

As the results just given speak for themselves, extended comment is unnecessary. The method herein described is based on the fundamental conditions of equilibrium and eliminates all assumptions as to the similarity between columns and beams. The difference between practical and purely theoretical cases has been explained and a means furnished by which engineers can design for shear in columns intelligently and with complete understanding of the significance of their assumptions. For purely theoretical deflection curves, it is not difficult to find the maximum shear corresponding to the bending allowances of the standard column formulas. It is highly important, however, to bear in mind that in practical problems the deflection curve may assume a shape which produces much greater maximum shear than the theoretical curve, without causing any increase of bending moment. The example shown in Fig. 2 illustrates such a condition.

A fixed-ended column is acted upon by end couples in addition to the applied load. These couples do not cause any shear because they produce constant bending moment and no shear results from a constant bending moment. For the purely theoretical deflection curve usually assumed, having points of contraflexure at the quarter-points, the maximum shear is found to be $\frac{4\,P\,f}{L}$, which is the same as that derived for the example represented by Fig. 1. In this case, however, the maximum bending moment is only $\frac{P\,f}{2}$, owing to the counter moment of the couples, so that the maximum shear for the purely theoretical deflection curve will be $\frac{8\,M}{L}$.

In this paper, the term "applied load" signifies the total force acting at the end of a column. If there is a lateral reaction in addition to the load supported (for example, the case of a column fixed at one end and hinged at the other), the applied load will be the resultant of the two forces.

An eccentric load does not produce any greater shear than is produced by a concentric load of the same amount, provided the eccentricity is the same at both ends of the column and provided the deflection curve is the same in the two columns. In the eccentrically loaded column the bending moment will be the sum of the bending moment which exists in the concentric column plus a constant bending moment due to eccentricity; but a constant bending moment does not produce any shear. Therefore, the shear in the two columns will be the same. If, however, the deflection is increased by the added bending moment due to eccentricity, the shear will be increased also, for shear is always equal to $P \sin G$.

In practical problems the eccentricity will ordinarily not be the same at the two ends, and lateral reactions will be introduced which must be considered in the computation for shear. Fig. 3 shows a column which bears on its left-hand edge at the top and on its right-hand edge at the bottom. P_1 is the load supported; P_2 is the lateral reaction, the value of which is $\frac{P_1 e}{L}$; and P_r is the resultant of the two forces. The shear at the middle of the column is $P_r \sin G_r$, but it is not necessary to compute the direction and amount of the resultant force, P_r, because the shear will be $P_2 + P_1 \sin G_1$ (very nearly). The error

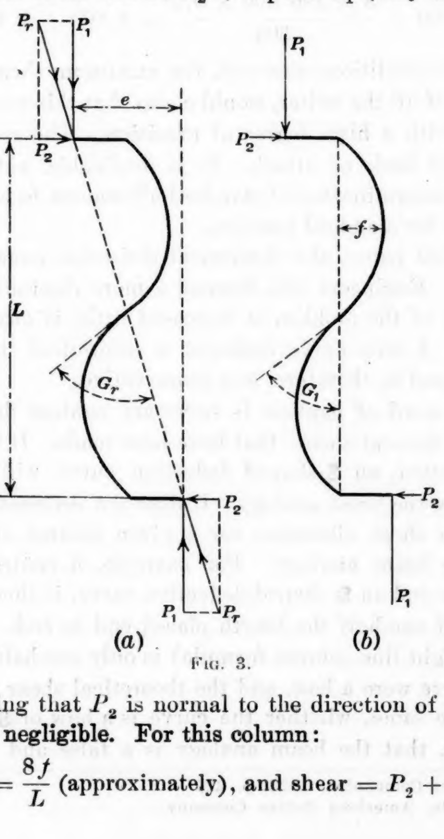


Fig. 3.

involved in assuming that P_2 is normal to the direction of the column axis at its middle point is negligible. For this column:

$$\sin G_1 = \frac{8f}{L} \text{ (approximately), and shear} = P_2 + \frac{8 P_1 f}{L}$$

As this condition is usually considered dangerous, but indeterminate, it will be interesting to examine it somewhat in detail. Let it be required to find the maximum shear in a standard latticed channel column 25 ft. long composed of two 10-in. by 30-lb. standard channels.* It will be assumed that the conditions are similar to Fig. 3, and that no allowance was made for eccentricity, as central loading was assumed and the eccentricity resulted from poor workmanship:

Safe total load, by the formula† = 180 000 lb. $\frac{P}{a} = 19\ 000 - 100\ \frac{L}{r}$

8 in. = value of e (Fig. 3), assumed.

$$P_2 \text{ (Fig. 3)} = 180\ 000 \times \frac{8}{300} = 4\ 800 \text{ lb.}$$

$$\text{Shear} = P_2 + \frac{8\ P_1 f}{L}$$

In order to avoid exaggeration, f will be assumed to be only 0.1 of 1% of the length. It was 0.22 of 1% for the member, *A-9-L,* of the first Quebec Bridge. Therefore:

$$\text{Shear} = 4\ 800 + \frac{8 \times 180\ 000 \times 0.3}{300} = 4\ 800 + 1\ 440 = 6\ 240 \text{ lb.}$$

Therefore, for the conditions assumed, the maximum shear would be 0.035 P_1.

No one, least of all the writer, would claim that this result represents actual field conditions with a high degree of precision. The method, however, does furnish a rational basis of attack. It is applicable, not so much to actual design, as to the determination of standard allowances to be included in specifications and rules for standard practice.

Throughout this paper, the theoretical deflection curve has been assumed to be a parabola. Engineers who demand a more rigorous treatment will find that the difficulty of the problem is increased little, if any, by the assumption of a sine curve. A sine curve indicates a theoretical shear about 20% less than a parabola, and is, therefore, less conservative.

In closing, a word of caution is necessary against drawing unwarranted conclusions from the statements that have been made. It has been stated that, for a given deflection, an **S**-shaped deflection curve will produce twice the shear indicated by the beam analogy. It does not necessarily follow from this, however, that the shear allowance for a given column should be twice that indicated by the beam analogy. For example, a centrally loaded column, having round ends and an **S**-shaped deflection curve, is theoretically equivalent to two columns of one-half the length placed end to end. The bending effect (assuming a straight-line column formula) is only one-half what it would be if the deflection curve were a bow, and the theoretical shear to be allowed for in such a case is the same, whether the curve is a bow or **S**-shaped. The fact, remains, however, that the beam analogy is a false and misleading concept.

* Carnegie Pocket Companion, 1923, p. 225.

† Column formula, American Bridge Company.

Applied to Member *A-9-L* of the first Quebec Bridge, it not only gives a result approximately one-half the true value, but also fails to show at what point in the member the maximum shear occurred. It gives results too small for columns designed as fixed-ended, and for all columns that develop the full contemplated bending moment at any point except the middle.

Acknowledgment is due to J. P. J. Williams, M. Am. Soc. C. E., for valuable suggestions in the preparation of this paper.

CO-ORDINATION OF IRRIGATION AND POWER*

By William Kelly,† M. Am. Soc. C. E.

Synopsis

Irrigation and power are both of great importance in the development of the Western States. Both interests require regulation of stream flow, but to different degrees, which may bring them into conflict. This paper gives a general presentation of the factors entering into the problem, illustrates the difficulties, and points out certain solutions by describing some of the specific cases that have come before the Federal Power Commission.

General

The co-ordination of irrigation and power is a problem found on nearly every stream in the Arid West. The Arid West is a region in which the mountains exercise a marked influence on climatic as well as on physical conditions. The annual precipitation in this region, except in the extreme Northwest, varies from 2 in. to about 15 in., whereas in the non-arid parts of the country the annual precipitation exceeds 25 in. The larger part of the precipitation in the West occurs on the mountain ranges often in the form of snow. Most of the streams get their principal supply from high up on the ranges, and the water in running down to the lower levels where farming is practicable offers opportunity for power development. The area west of the Mississippi River contains about 72% of the potential water power of the United States.

At present there is a well-established principle in the Western States that the use of water for irrigation is superior to any other except domestic use, and the laws of those States reflect that principle. It is not the purpose of this paper to discuss the merits of that principle. States have a right to establish such principles and the Federal Government always has and always will recognize that right so long as it does not infringe on the rights of others under the Federal Constitution.

The principle must be borne in mind, however, in considering co-ordination of irrigation and power because to a considerable extent it sets aside the laws of economics and often causes co-ordination to take the form of adjusting power development so that it will not curtail irrigation development. It should also be borne in mind that, as a rule, power development has proved more profitable to investors than irrigation, and is, therefore, more easily financed.

* Presented at the Summer Meeting, Salt Lake City, Utah, July 8, 1925.

† Director of Eng., National Electric Light Assoc., New York, N. Y.

While irrigation is often seeking State or Federal aid, power does not need such aid; on the contrary, it frequently gives aid to irrigation without recompense.

Development of storage to regulate flow is more likely to be financially practicable for a power project than for an irrigation project, and most of the successful irrigation projects that have developed storage have produced power as a by-product to help carry the cost of the storage. The irrigation use of water is distinctly a seasonal use. With a few exceptions, such as the Imperial Valley, the season lasts from four to nine months and in most cases the peak demand occurs in June and July. Fortunately for farming, the maximum flow of most Western rivers being due to melting snows, occurs at the time of the maximum irrigation demand, thus reducing materially the storage capacity needed.

The power use of the water, on the other hand, is fairly uniform throughout the year, and, therefore, requires sufficient storage for more or less complete seasonal regulation. Secondary or seasonal power has a limited value. To the extent that power can be used for irrigation pumping, the power demand for water can be made to conform to the irrigation demand. Generally, the shortage of water for irrigation occurs in the latter part of the irrigation season; storage for power increases the discharge during this season, and to that extent is beneficial to irrigation. Figs. 1 to 6, inclusive, show graphically the relation between the irrigation demand and the monthly discharge for a high, average, and low-water year on the following rivers: Colorado, Columbia, Snake, San Joaquin and principal tributaries, Sacramento and principal tributaries, and Kings.

Examination of Fig. 1, Colorado River, shows that there is not much need for storage for irrigation for the Lower Basin provided it is practicable to keep the demand down to 5 acre-ft. per acre per year at the point of diversion. For the Upper Basin, however, storage is essential to provide for the irrigation demand from July to October, inclusive. The irrigation demand and its distribution through the year is taken from an unpublished report by the United States Bureau of Reclamation.

Examination of Fig. 2, Columbia River, indicates that the stream flow is more than ample to take care of irrigation at any and all times. As explained later, however, the irrigable lands lie so high above the main river that gravity diversion is possible only on the tributaries at points where the discharge is not adequate without storage.

Fig. 3 shows that storage for irrigation on Snake River is needed from June to September, inclusive. The Snake River in Idaho presents an interesting example of co-ordinated irrigation and power. (See Fig. 7.) At several places on the Snake all the low-water flow is diverted for irrigation, and yet a short distance below such diversion the river flow is revived by underground contributions. In Southern Idaho the river flows through a lava formation and between the more or less horizontal layers of lava are water-bearing strata. Diversion for irrigation greatly exceeds consumptive use and much of the water diverted finds its way back to the river.

Storage has been developed at Jackson Lake, Wyoming, and Lake Walcott, Idaho, and is being developed at American Falls, Idaho. The irrigation developments have been made both by districts organized under the State law and by the U. S. Bureau of Reclamation. Power has been developed by the U. S. Bureau of Reclamation and by the Idaho Power and Light Company. The power is practically all used for irrigation pumping and for domestic purposes

TOTAL DEMAND

Irrigable Area - Lower Basin 1 000 000 Acres at 5.0 Acre Ft. = 5 000 000 Acre Ft.
Irrigable Area - Upper Basin 4 000 000 Acres at 1.5 Acre Ft. = 6 000 000 Acre Ft.

MONTHLY DEMAND

		Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Total
Percentage	Lower Basin	4	5	8	10	11	12	12	11	10	8	5	4	100
	Upper Basin	−3⅓	−3⅓	−3⅓	−3⅓	10	20	33⅓	33⅓	20	6⅔	−6⅔	−6⅓	100
Thousands of Acre Feet.	Lower Basin	200	250	400	500	550	600	600	550	500	400	250	200	5000
	Upper Basin*	−200	−200	−200	−200	600	1200	2000	2000	1200	400	−400	−200	6000
	Net Demand	0	50	200	300	1150	1800	2500	2550	1700	800	−150	0	11 000

*Includes Return Flow

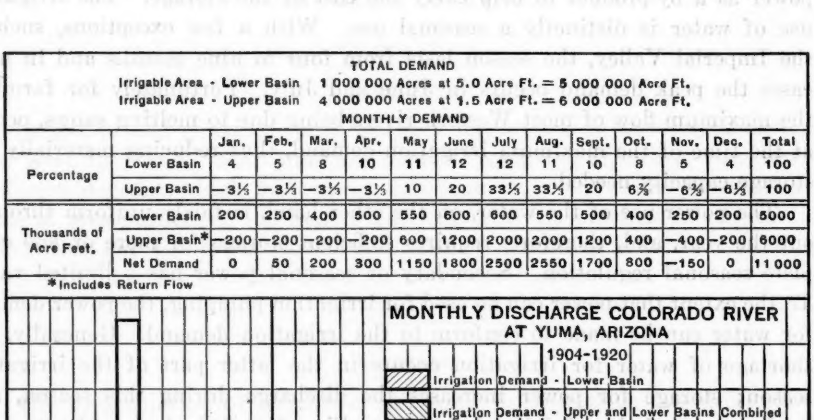


FIG. 1.

by the communities supported by the irrigation projects. In this part of Idaho, farming and stock-raising are practically the only industries. In such a situation there can be no serious conflict over the relative use of water for irrigation and power, because the market for power depends on the success and extent of the irrigation developments, and power is economically and practically secondary to irrigation.

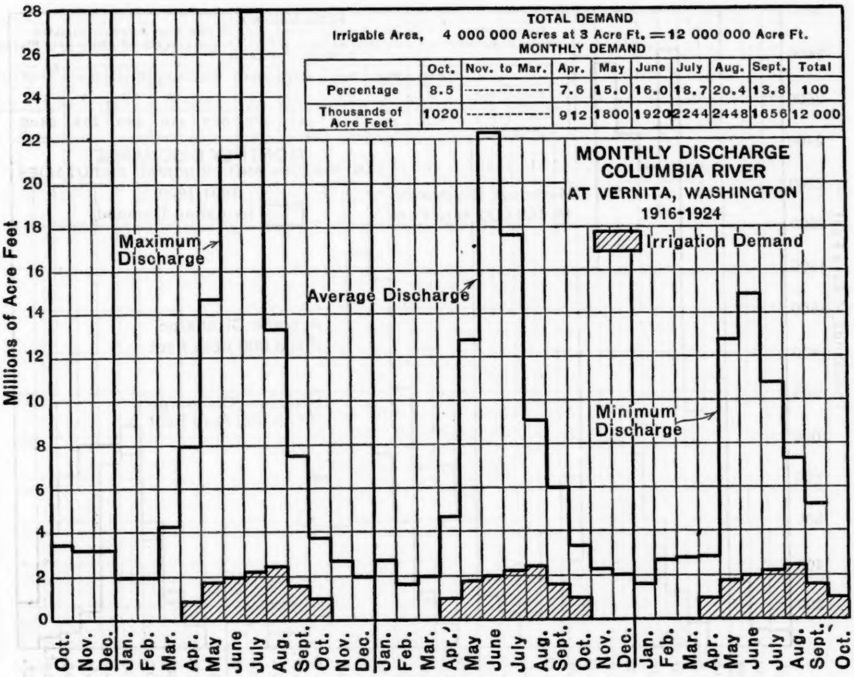

TOTAL DEMAND
Irrigable Area, 4 000 000 Acres at 3 Acre Ft. = 12 000 000 Acre Ft.
MONTHLY DEMAND
Oct. | Nov. to Mar. | Apr. | May | June | July | Aug. | Sept. | Total
Percentage | 8.5 | ------------ | 7.6 | 15.0 | 16.0 | 18.7 | 20.4 | 13.8 | 100
Thousands of Acre Feet | 1020 | ------------ | 912 | 1800 | 1920 | 2244 | 2448 | 1656 | 12 000
MONTHLY DISCHARGE
COLUMBIA RIVER
AT VERNITA, WASHINGTON
1916-1924
Irrigation Demand
Maximum Discharge
Average Discharge
Minimum Discharge
Millions of Acre Feet
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
Oct.
Nov.
Dec.
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.

FIG. 2.

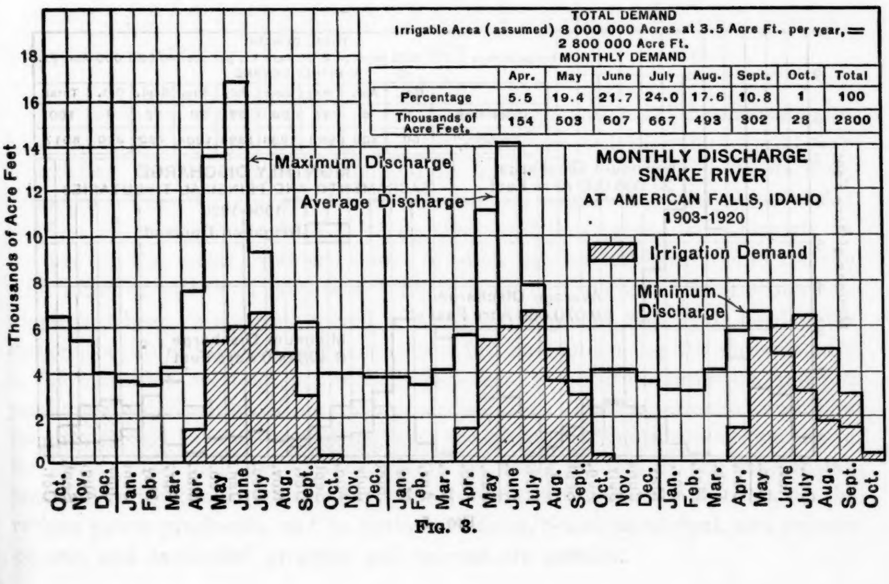

TOTAL DEMAND
Irrigable Area (assumed) 8 000 000 Acres at 3.5 Acre Ft. per year, =
2 800 000 Acre Ft.
MONTHLY DEMAND
Apr. | May | June | July | Aug. | Sept. | Oct. | Total
Percentage | 5.5 | 19.4 | 21.7 | 24.0 | 17.6 | 10.8 | 1 | 100
Thousands of Acre Feet. | 154 | 503 | 607 | 667 | 493 | 302 | 28 | 2800
MONTHLY DISCHARGE
SNAKE RIVER
AT AMERICAN FALLS, IDAHO
1903-1920
Irrigation Demand
Maximum Discharge
Average Discharge
Minimum Discharge
Thousands of Acre Feet
0
2
4
6
8
10
12
14
16
18
Oct.
Nov.
Dec.
Jan.
Feb.
Mar.
Apr.
May
June
July
Aug.
Sept.

FIG. 3.

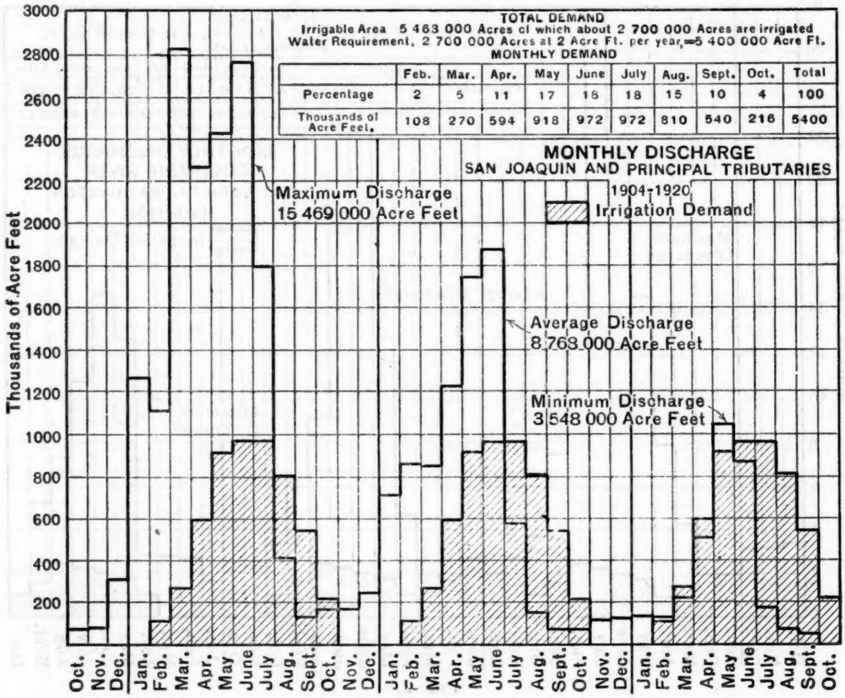

TOTAL DEMAND
Irrigable Area 5 463 000 Acres of which about 2 700 000 Acres are irrigated
Water Requirement, 2 700 000 Acres at 2 Acre Ft. per year,=5 400 000 Acre Ft.
MONTHLY DEMAND
Feb. Mar. Apr. May June July Aug. Sept. Oct. Total
Percentage 2 5 11 17 18 18 15 10 4 100
Thousands of Acre Feet. 108 270 594 918 972 972 810 540 216 5400
MONTHLY DISCHARGE
SAN JOAQUIN AND PRINCIPAL TRIBUTARIES
1904-1920
Irrigation Demand
Maximum Discharge
15 469 000 Acre Feet
Average Discharge
8 763 000 Acre Feet
Minimum Discharge
3 548 000 Acre Feet
Thousands of Acre Feet
3000
2800
2600
2400
2200
2000
1800
1600
1400
1200
1000
800
600
400
200
Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct.

FIG. 4.

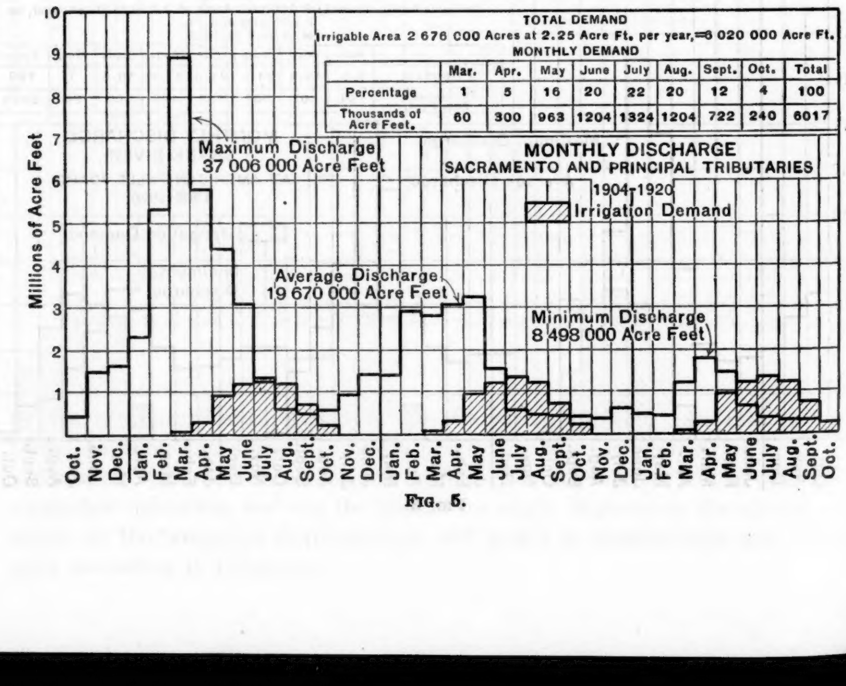

TOTAL DEMAND
Irrigable Area 2 676 000 Acres at 2.25 Acre Ft. per year,=6 020 000 Acre Ft.
MONTHLY DEMAND
Mar. Apr. May June July Aug. Sept. Oct. Total
Percentage 1 5 16 20 22 20 12 4 100
Thousands of Acre Feet. 60 300 963 1204 1324 1204 722 240 6017
MONTHLY DISCHARGE
SACRAMENTO AND PRINCIPAL TRIBUTARIES
1904-1920
Irrigation Demand
Maximum Discharge
37 006 000 Acre Feet
Average Discharge
19 670 000 Acre Feet
Minimum Discharge
8 498 000 Acre Feet
Millions of Acre Feet
10
9
8
7
6
5
4
3
2
1
Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct. Nov. Dec. Jan. Feb. Mar. Apr. May June July Aug. Sept. Oct.

FIG. 5.

Fig. 4 shows that the water supply in the San Joaquin Valley, California, is deficient generally from July to October, inclusive, for the area now irrigated, which is only about one-half the area that would be irrigable if the water could be found. The deficiency is more or less overcome by pumping from the underground supply. Storage in connection with power developments has aided by building up the flow during the period of deficiency. Storage by irrigation interests has been undertaken recently on the Tuolumne River and is under construction on the Merced. Such projects are under consideration also on the Mokelumne, Stanislaus, and Kings Rivers. A more detailed description of the Kings River situation follows.

Present Irrigated Area, 700 000 Acres at 2 Acre Ft. = 1 400 000 Acre Ft.

DESIRABLE MONTHLY DEMAND PER STATE ENGINEER

	Feb.	Mar.	April	May	June	July	Aug.	Sept.	Oct.	Total
Percentage	2	5	11	17	18	18	15	10	4	100
Thousands of Acre Feet	28	70	154	238	252	252	210	140	56	1400

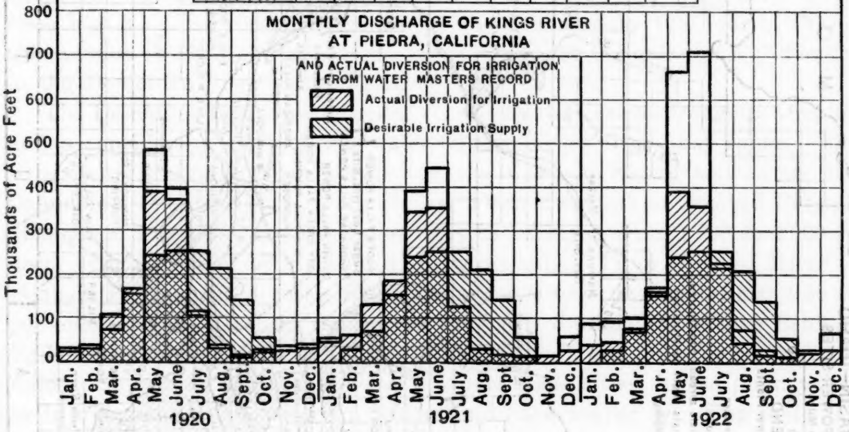


FIG. 6.

Fig. 5 shows that there is more than enough water in the Sacramento River to irrigate all the land suitable for irrigation, but that the natural flow is deficient for the irrigation demand in the months of July to September, inclusive. The Sacramento River presents an interesting and complicated problem of conflicting interests of up-river and down-river irrigationists in the use of the water, and the effect of such use on navigation and on the infiltration of salt from the ocean. There is also a problem of flood protection which has been fairly well solved. Power development requiring head-water storage, building up the late summer and fall flow and using the streams only in the mountain sections, is generally beneficial to all other interests, and not particularly difficult to bring into co-ordination. The Sacramento and San Joaquin Rivers have dependent on them the greatest irrigation development in the world. The problems of these rivers are being studied by the substantial, practical irrigators to whose efforts the present developments are due, by the various power producers, and by various Federal, State, municipal, and private experts, and continued progress and success are certain.

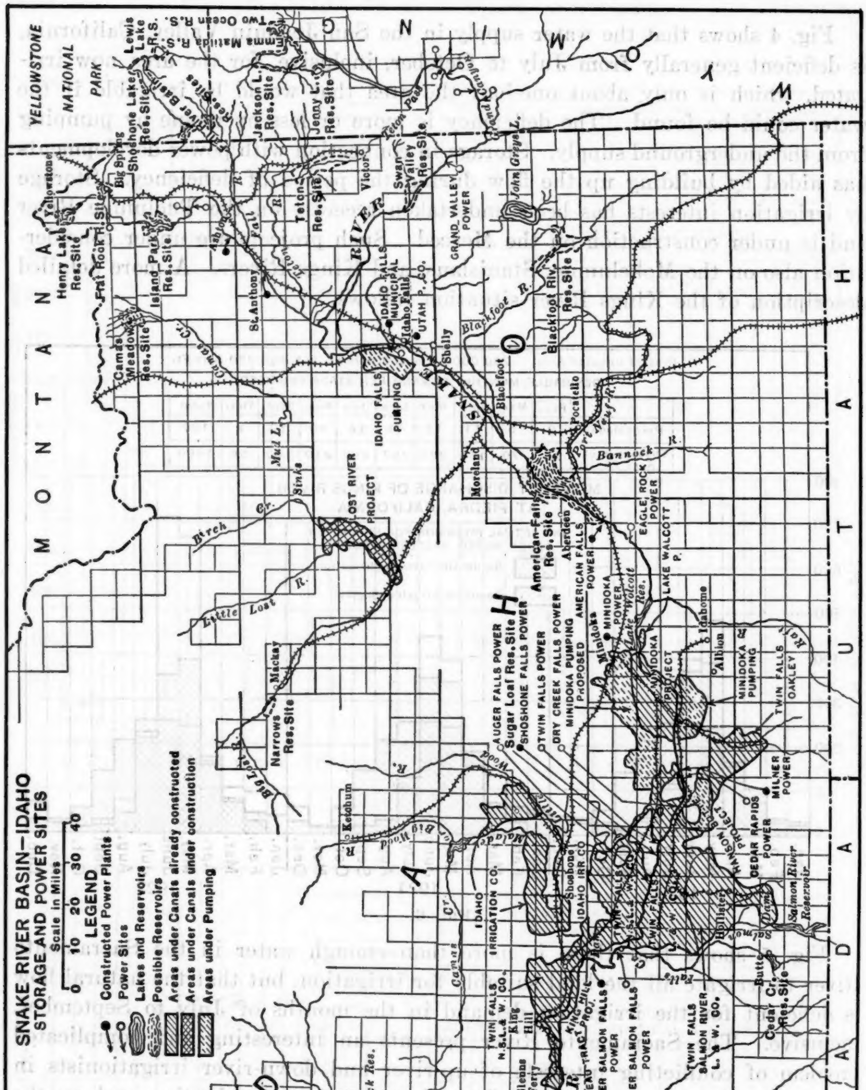

SNAKE RIVER BASIN—IDAHO
STORAGE AND POWER SITES
LEGEND
Scale in Miles
0 10 20 30 40
Constructed Power Plants
Power Sites
Lakes and Reservoirs
Possible Reservoirs
Areas under Canals already constructed
Areas under Canals under construction
Areas under Pumping
M O N T A N A
W Y O M I N G
U T A H
N E V A D A
O R E G O N
I D A H O
YELLOWSTONE NATIONAL PARK
SNAKE RIVER
BOISE
Twin Falls
Idaho Falls
Pocatello
Blackfoot
American Falls
Mountain Home
Huntington

FIG. 7.

Perhaps the best way to bring out in detail the difficulties encountered in co-ordinating irrigation and power will be to describe some of the problems of this kind that have come before the Federal Power Commission. Nearly every stream presents a special problem of its own.

In connection with certain applications for power development the Federal Power Commission found a need of comprehensive studies to determine the relations between irrigation and power on the following streams: The Deschutes River, in Oregon; the Columbia River, in Washington, Idaho, and Montana; and the Trinity, American, and Stanislaus Rivers, in California.

Deschutes River, Oregon

A Board consisting of J. B. Cavanaugh, Colonel, Corps of Engineers, U. S. A., D. C. Henny, M. Am. Soc. C. E., Consulting Engineer, U. S. Bureau of Reclamation, and F. F. Henshaw, M. Am. Soc. C. E., District Engineer, U. S. Geological Survey, was appointed to report on the Deschutes River. (See Fig. 8.) The report of the Board was published by the Federal Power Commission in 1922, and has shaped the policy of the Commission with respect to the Deschutes River since that time.

The Deschutes Basin lies immediately east of the Cascade Range in Oregon, and a considerable part of the water supply comes from that range. The river flows generally north paralleling the Cascade Range and enters the Columbia a few miles east of The Dalles. From its mouth to Crooked River, a distance of 112 miles, the Deschutes flows in a canyon from 1 000 to 2 000 ft. deep, with a river slope of about 12.8 ft. per mile. From Crooked River to Benham Falls, about 70 miles, the slope increases to an average of about 38 ft. per mile, and the depth of the canyon gradually diminishes. Above Benham Falls the gradient suddenly flattens to about 1.5 ft. per mile for the next 36 miles. Storage is practicable above Benham Falls either in the main valley or in numerous small mountain valleys and lakes higher up. The rapid fall below Benham Falls and the relative shallowness of the canyon for about 20 miles down stream render irrigation diversion comparatively easy, and all present gravity diversions take place in this stretch of the river. Diversions more than 25 miles below Benham Falls are impracticable on account of the depth of the canyon.

The flow of the Deschutes River is remarkably uniform. At the mouth, it generally fluctuates between 5 000 and 8 000 sec-ft. The lowest flow during the last 14 years was about 3 700 sec-ft., and short-time flood peaks usually do not exceed 16 000 sec-ft. This is due to the very large inflow of underground water from the porous volcanic formation above Crooked River as well as to the well-sustained low-water surface flow from the high Cascades. The minimum monthly mean flow of the Deschutes at Metolius, Ore., is 3 250 sec-ft. This minimum will not be materially affected if all the surface water reaching Benham Falls is diverted for irrigation. If all Upper Deschutes water is dedicated to irrigation, the present irrigated area of about 81 000 acres can be expanded to 191 000 acres, and 555 000 h.p. can be developed on the Lower Deschutes.

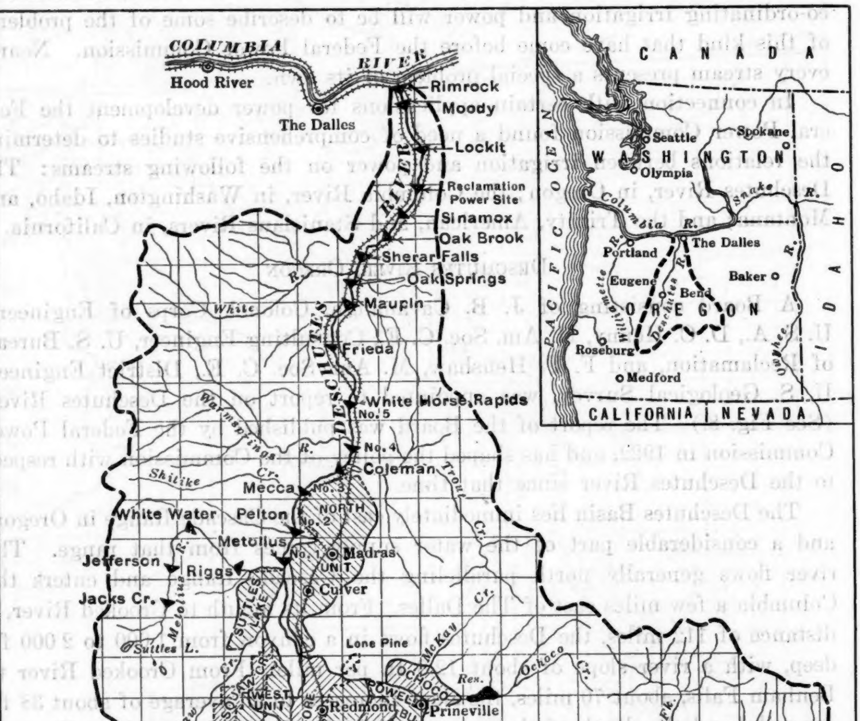

COLUMBIA
RIVER
Hood River
The Dalles
Rimrock
Moody
Lockit
Reclamation
Power Site
Sinamox
Oak Brook
Sherar Falls
Oak Springs
Maupin
Frieda
White Horse Rapids
No. 5
No. 4
Coleman
Mecca
No. 3
No. 2
No. 1
White Water
Pelton
Metolius
Jefferson
Riggs
Jacks Cr.
Madras
NORTH UNIT
Culver
Suttles L.
Lone Pine
Redmond
Prineville
OCHOCO
Ochoco
McKay Cr.
Res.
WEST UNIT
NORTH CANAL UNIT
CENT. ORE
SOUTH UNIT
ARNOLD
Bend
CROOKED RIVER
Post
Post Res. Site
North Fk.
Beaver Cr.
Camp Cr.
Bear Cr.
Green L.
Tumalo Cr.
Squaw Cr.
Shitike Cr.
Warm Springs R.
White R.
Trout Cr.
Dry River Bed
Crane Prairie Res. Site
Benham Falls Res. Site
Paulina L.
Walker Basin
Deschutes R.
Davis L.
Odell L.
Crescent Cr.
Crescent L.
East Fk.
DESCHUTES R.
CANADA
WASHINGTON
Seattle
Spokane
Olympia
Columbia R.
Snake R.
PACIFIC OCEAN
Portland
The Dalles
Eugene
Bend
Baker
OREGON
Roseburg
Medford
IDAHO
CALIFORNIA
NEVADA
DESCHUTES RIVER OREGON
IRRIGATION AND POWER SITES
Scale of Miles
0 5 10 15 20 25
LEGEND
Power Sites
(Numbers Refer to Application of Columbia Valley Power Co.)
Lands Irrigated
Lands Previously Proposed for Irrigation

FIG. 8.

There are at present six small hydro-electric developments on the Deschutes, all above the mouth of Crooked River. Their aggregate installed capacity is 3 230 h.p. The water rights of these developments conflict with irrigation expansion and will have to be extinguished for full irrigation development. The expansion of irrigation is likely to be delayed by the fact that all the irrigable areas are more than 4 000 ft. above sea level, so that only the hardier crops can be grown, and also by the fact that much of the area is now used more or less successfully for dry farming. This river presents an unusual condition in that all the irrigable areas lie above the main fall in the stream, and the peculiar physical and geological formation makes it possible to develop practically all the power in the lower river without curtailing irrigation development. Much of the power in the lower river can be developed at a very low cost and it undoubtedly will be developed whenever a market can be found.

On June 25, 1924, the Federal Power Commission issued a license to the Columbia Valley Power Company for developments at the Pelton and Metolius, Ore., sites on Deschutes River, with an ultimate installed capacity of 185 000 h.p. The Company proposed to sell power to the utility companies in Portland, Ore.

On November 28, 1922, the Federal Power Commission issued a preliminary permit to the Pacific Power and Light Company for a development at the Reclamation site on Deschutes River, with a proposed installed capacity of 54 000 h.p.

On November 13, 1924, the Federal Power Commission issued a preliminary permit to the Pringle Falls Electric Power and Water Company for a project near Bend, Ore., with a proposed installed capacity of 5 400 h.p.

No other applications are now before the Commission for developments on the Deschutes.

Columbia River

A Board consisting of J. B. Cavanaugh, Colonel, Corps of Engineers, U. S. A., D. C. Henny, M. Am. Soc. C. E., Consulting Engineer, U. S. Bureau of Reclamation, F. F. Henshaw, M. Am. Soc. C. E., District Engineer, U. S. Geological Survey, C. S. Heidel, M. Am. Soc. C. E., State Engineer of Montana, W. G. Swendsen, Assoc. M. Am. Soc. C. E., Commissioner of Reclamation, Idaho, and Marvin Chase, M. Am. Soc. C. E., Supervisor of Hydraulics, State of Washington, was appointed to make a study of the Columbia River (see map, Fig. 9), from Flathead Lake, Montana, to the mouth of Snake River; to report on the character and extent of present and prospective uses of the river for navigation, irrigation, and power, with a discussion of the relative merits of each and of their relation to each other; and to outline a program of development that would harmonize the conflicting uses in such manner as to secure, in the long run, the greatest combined benefit from all.

After nine months' investigation, the Board submitted a report which was published by the Federal Power Commission in 1923. The report was concurred in by all members of the Board except the representative of Washington, who concurred in all conclusions and recommendations except those with respect to the Columbia Basin project.

The Columbia drains an area of about 259 000 sq. miles segregated by stream basins, as follows:

	Square Miles.	Percentage.
Columbia River above Clark Fork in Canada..	34 000	13.1
Clark Fork	26 000	10.0
International Boundary to Snake River........	43 000	16.7
Snake River	109 000	42.1
Columbia River below Snake River.........	47 000	18.1
Total..	259 000	100.0

The mean annual precipitation of the Columbia Basin varies according to location, from 6 to more than 100 in. Where the precipitation exceeds 20 in. the country is generally forested and mostly mountainous, and where it is less than 15 in. the country is arid. The areas with low rainfall lie in two broad belts, one comprising East-Central Washington and extending northward up the Okanogon Valley into British Columbia, the other lying in the inter-mountain valley of Montana and British Columbia. Most of the land sufficiently smooth for agricultural development lies in the arid area.

The mean annual discharge at various points from Flathead Lake to the mouth of the Snake River is shown in Table 1.

TABLE 1.

Station.	Mean annual discharge, in acre-feet.	Drainage area, in square miles.
Flathead River, Polson, Mont..................	8 108 800	7 200
Clark Fork, Plains, Mont.......................	14 914 400	19 900
Clark Fork, Metaline Falls, Wash.............	19 982 400	25 100
Columbia River, International Boundary.....	72 841 200	60 000
Columbia River, Pasco, Wash..................	94 699 200	103 000

The Upper Basin of the Columbia abounds in lakes, nearly all of which present possibilities of water storage. The three principal lakes are:

Flathead Lake, in Western Montana, at an elevation of nearly 2 900 ft., which has a surface area of 120 000 acres. The lake has a fairly steep shore except at its upper end where the Upper Flathead River has built up a large delta. A considerable part of this delta is flooded at ordinary high water, but the remainder is used for agriculture which constitutes the principal support of the country around Kalispell, Mont. The outlet of the lake is through a rocky gorge below Polson offering satisfactory foundation for a dam. Storage to the amount of 1 600 000 acre-ft. can be obtained without serious damage from flooding.

Lake Pend Oreille, in Northern Idaho, has a surface area of 86 000 acres at low water and 98 000 acres at high water. The shores are generally steep, but much of the western portion is bordered by gravel beaches. The meadow lands at the head of the lake have the characteristics of a delta which the river overflows in flood. The outlet of the lake is between rocky banks and at Albany Falls the river crosses a dike of hard rock affording satisfactory foundation for a controlling dam. A storage capacity of 1 750 000 acre-ft. can be obtained without serious damage from flooding.

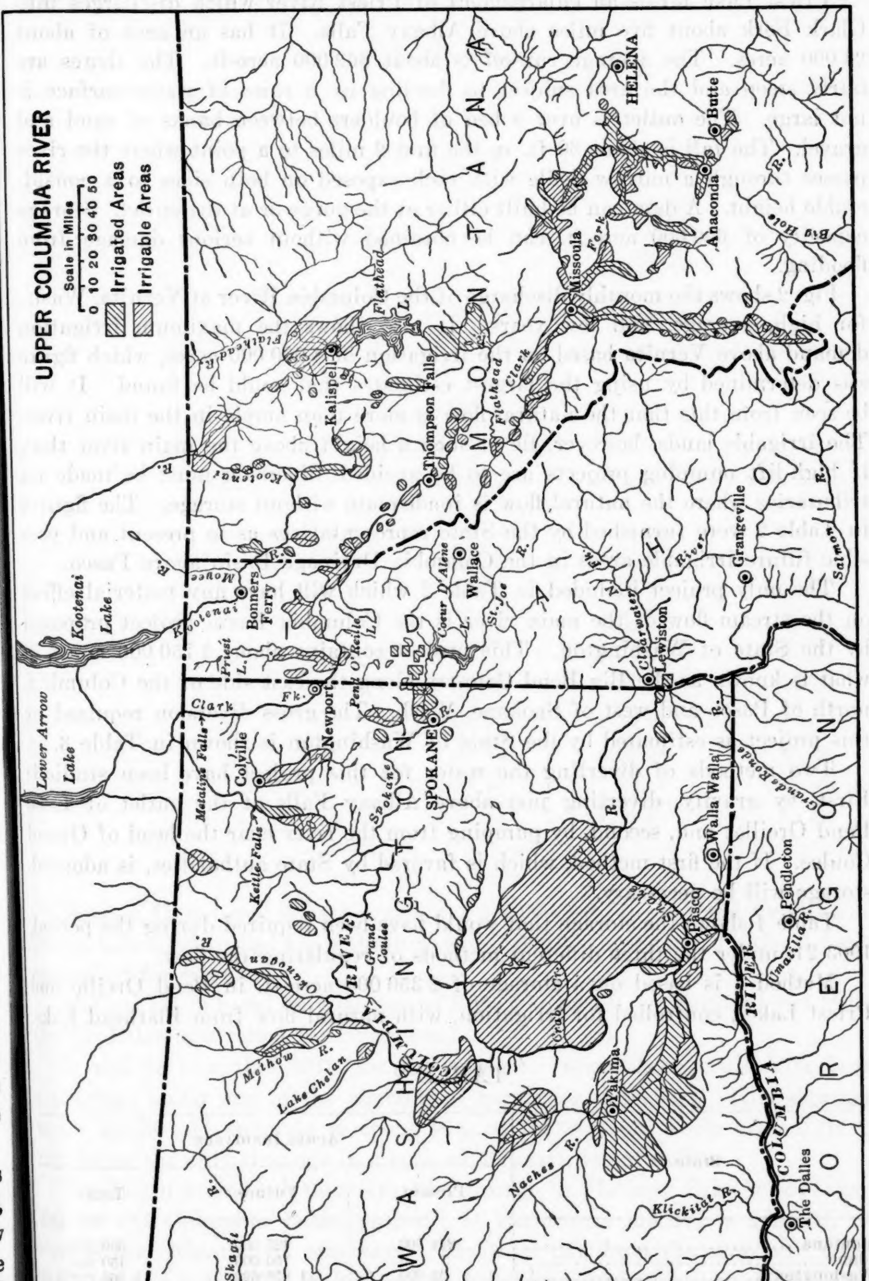

UPPER COLUMBIA RIVER
Scale in Miles
0 10 20 30 40 50
Irrigated Areas
Irrigable Areas
Lower Arrow Lake
Kootenai Lake
Kootenai R.
Bonners Ferry
Priest L.
Clark Fk.
Metaline Falls
Colville
Newport
Pend O'reille L.
Kettle Falls
SPOKANE
Spokane R.
Coeur d'Alene
Wallace
St. Joe R.
Thompson Falls
Kalispell
Flathead L.
Flathead R.
Kootenai R.
Missoula
HELENA
Butte
Anaconda
Big Hole R.
Clark Fk.
Lewiston
Clearwater R.
Grangeville
Salmon R.
Walla Walla
Pendleton
Umatilla R.
Grande Ronde
Pasco
Yakima
Naches R.
Klickitat R.
The Dalles
COLUMBIA RIVER
Grand Coulee
Okanogan R.
Methow R.
Lake Chelan
Skagit R.
Crab Cr.
WASHINGTON
IDAHO
MONTANA
OREGON

FIG. 9.

Priest Lake forms an enlargement of Priest River which discharges into Clark Fork about five miles above Albany Falls. It has an area of about 24 000 acres. The average run-off is about 862 000 acre-ft. The shores are fairly steep and the area subject to flooding by a raise of water surface is not large. The outlet is over a bed of boulders between banks of sand and gravel. The fall is about 30 ft. in the first 2 miles to a point where the river passes through a narrow defile with rock exposed on both sides to a considerable height. A dam can be built either at the gorge or at the outlet. Storage capacity of 600 000 acre-ft. can be obtained without serious damage from flooding.

Fig. 2 shows the monthly discharge of the Columbia River at Vernita, Wash., for high, average, and low years. It also shows the maximum irrigation demand above Vernita based on the irrigation of 4 000 000 acres, which figure was determined by using the largest estimates that could be found. It will be seen from this that the natural flow is more than ample in the main river. The irrigable lands, however, lie at such a height above the main river that, if high-lift pumping projects are to be avoided, diversion must be made on tributaries where the natural flow is inadequate without storage. The figures in Table 2 were furnished by the State representatives as to present and possible future irrigable areas in the Columbia Drainage Basin above Pasco.

The only project included in Table 2 which will have any material effect on the stream flow of the main river is the Columbia Basin project proposed by the State of Washington. This project contains about 1 750 000 acres in what is known as the Big Bend Country along the east side of the Columbia, north of Pasco and west of Spokane, Wash. The gross diversion required by this project as estimated by the State of Washington is shown in Table 3.

Two methods of diverting the water for this project have been studied: First, by gravity, diverting just above Albany Falls at the outlet of Lake Pend Oreille; and, second, by pumping from the river near the head of Grand Coulee. If the first method, which is favored by State authorities, is adopted, storage will be necessary.

Table 4 shows the storage that would have been required during the period, 1903-21, under the three different methods of regulating the flow.

Method I is based on a storage of 2 350 000 acre-ft. in Pend Oreille and Priest Lakes, controlled for irrigation, with natural flow from Flathead Lake.

TABLE 2.

State.	Acres Irrigable.		
	Present.	Future.	Total.
Montana	274 000	425 000	699 000
Idaho	0	150 000	150 000
Washington	95 000	1 870 622	1 965 622
Total	369 000	2 445 622	2 814 622

TABLE 3.

Month.	Acre-feet.
April	561 000
May	152 000
June	1 152 000
July	152 000
August	1 092 000
September	685 000
October	436 000
Total	6 228 000

TABLE 4.

Year.	Method I.	Method II.	Method III.
1903	649 000	1 005 000	1 118 000
1904	1 584 000	1 451 000	1 830 000
1905	2 195 000	2 389 000	2 603 000
1906	2 185 000	2 083 000	2 561 000
1907	160 000	225 000	349 000
1908	1 098 000	1 011 000	1 451 000
1909	965 000	1 132 000	1 388 000
1910	1 914 000	2 091 000	2 310 000
1911	1 226 000	1 211 000	1 461 000
1912	649 000	822 000	823 000
1913	846 000	828 000	1 118 000
1914	1 568 000	1 329 000	1 695 000
1915	1 124 000	1 162 000	1 362 000
1916	21 000	50 000	78 000
1917	1 046 000	463 000	1 508 000
1918	1 505 000	1 389 000	1 678 000
1919	2 787 000	2 706 000	3 360 000
1920	1 080 000	1 165 000	1 348 000
1921	1 593 000	1 515 000	2 051 000

Method II is based on a storage of 2 350 000 acre-ft. in Pend Oreille and Priest Lakes, controlled for irrigation, with 1 600 000 acre-ft. in Flathead Lake, controlled for power.

Method III is based on a storage of 3 350 000 acre-ft. in Flathead and Pend Oreille Lakes controlled for irrigation.

It will be seen that 1919 was the only year when there would have been a shortage under any of the methods of regulation described. This shortage under Method II amounts to about 10% of the seasonal needs after July 1, and, being the only shortage in a cycle of 19 years, is not serious.

The Board concluded, therefore, that storage in Flathead Lake is not essential for the Columbia Basin project. If the second method is adopted, no storage for irrigation is necessary. The total lift from the river to Coulee is 630 ft. and a dam 200 ft. high would leave a remaining height of 430 ft. as pump lift. There is a good dam site at Grand Coulee with bed-rock at 60 ft. below low water. A pumping lift of 430 ft. is considerably greater than that of any successful pumping project now in existence, and its economic feasibility has been challenged. Whether or not economically feasible, a study

of the pumping project made for the State by Mr. W. T. Batcheller, Electrical and Mechanical Engineer, Seattle, Wash., shows that the cost per acre of a pumping project would be less than that of a gravity project, and this fact was confirmed by estimates made by the Columbia River Board.

A new study of the Columbia Basin project is being made by the U. S. Department of the Interior, the results of which are not yet available. All authorities appear to agree that the soil and climatic conditions are excellent, but there is a wide difference of opinion as to the probable costs. The costs have been variously estimated at from $145 to $240 per acre for the water system, including storage, diversion, main canals, main laterals, and distribution system.

The fall in the river from Flathead Lake to Pasco is 2 567 ft. Data with respect to dam sites are not complete, but based on the dam sites that look feasible, 1 398 ft. of the fall can be developed for power. Table 5 shows the all-year power possibilities under various assumed conditions.

From Table 5 it is evident that a decision between the methods of irrigating the Columbia Basin affects not only the project itself, but also future power possibilities. The selection of the gravity method will destroy nearly 650 000 all-year effective horse-power.

There is a tendency to under-estimate the value of power in a region such as the Northwest where the potential power is great and present demand relatively small. The demands of a rapidly increasing power market are being felt all over the United States. Twenty years ago California had a great excess of water power, but it is now evident that before many years additional sources of power will have to be sought, and the Northwest seems certain to follow the same trend.

The conclusions of the Columbia River Board were that:

(*a*) Freedom should be given to fullest irrigation expansion in Montana, Idaho, and Washington, and no rights should be allowed to accrue to the lower interests, which would legalize limitation of or interference with irrigation above.

(*b*) The Columbia Basin project is the most important single item to be considered in the uses to be made of Columbia River water above the mouth of Snake River.

(*c*) The Columbia Basin project can be irrigated by gravity from the Clark Fork at Albany Falls, or by pumping from the Columbia River at Grand Coulee.

(*d*) The Columbia Basin gravity project can be supplied adequately by the aid of storage in Pend Oreille and Priest Lakes in Idaho, conditioned on the maintenance of natural flow from Flathead Lake when needed.

(*e*) The Columbia Basin project can be supplied with practically no shortage without the aid of storage, with a dam at Grand Coulee about 156 ft. high above low water.

(*f*) Information on which to base a final decision between a gravity and a pumped supply for a Columbia Basin irrigation project is not complete and should be completed. Any decision should take into account the effect on potential power.

(*g*) Pending such decision no permanent rights of storage should be granted in Pend Oreille and Priest Lakes.

TABLE 5.—ALL-YEAR POWER ON FLATHEAD RIVER, CLARK FORK, AND COLUMBIA RIVER.

Stream.	Location.	Usable head at dam sites, in feet.	No storage. All year horse-power.	FLATHEAD STORAGE, 1 600 000 ACRE-FT. FOR POWER; PEND OREILLE AND PRIEST, 2 300 000 ACRE-FT. FOR: Columbia Basin gravity project, all-year horse-power.	Power, all-year horse-power.
Flathead River:					
Clark Fork..	Flathead Lake to mouth of Flathead River.......	303	69 000	165 000	165 000
No. 1.........	Mouth of Flathead River to Idaho line..........	140	75 000	106 000	106 000
No. 2.........	Idaho line to Lake Pend Oreille............	77	43 000	59 000	59 000
No. 3.........	Lake Pend Oreille to International Boundary..	250	232 000	176 000	435 000
No. 4.........	International Boundary to mouth of Clark Fork.	265	234 000	178 000	440 000
Columbia River:					
No. 1.........	Mouth of Clark Fork to foot of Kettle Falls.....	75	166 000	178 000	225 000
No. 2.........	Foot of Kettle Falls to Grand Coulee........	198	612 000	640 000	665 000
No. 3.........	Grand Coulee to foot of Priest Rapids.........	90	294 000	294 000	348 000
Total..................................		....	1 725 000	1 796 000	2 443 000

(*h*) If ultimately the decision is in favor of a Columbia Basin gravity project, storage rights in Pend Oreille and Priest Lakes should be granted to such project, but should be limited to storage of inflow in excess of 7 000 sec-ft.

(*i*) If ultimately the decision is in favor of a Columbia Basin pumping project, storage rights in Pend Oreille and Priest Lakes should be granted to the joint interests of power on the Lower Clark Fork and the Columbia River subject to limitations contained in Section (*a*).

(*j*) Storage rights in Flathead Lake should be granted to Flathead power interests, subject to the fullest development found to be practicable after complete investigation. Prior to such development, rights may be advantageously granted to power interests on the Clark Fork subject to conditions protecting ultimate control for Flathead power and natural flow release when needed for the protection of the Columbia Basin project if built on the gravity plan.

(*k*) Storage control at Flathead, Pend Oreille, and Priest Lakes should be under impartial supervision.

(*l*) A permit to develop power at the Grand Coulee site, or at any point on the Columbia River as far down stream as the Foster Creek site, should not be granted to power interests until it is known that such site will not be needed by the Columbia Basin project. In case the Columbia Basin project shall require a power site for irrigation and power on this stretch of the Columbia River, a permit should be granted to it after the best location and height of dam shall have been determined from the standpoint of the project and the public interest.

These conclusions appear to be sound and equitable and are, therefore, likely to be followed by the Federal Power Commission.

The following is the status of applications to the Federal Power Commission for power projects on the Upper Columbia:

Application for preliminary permit by the Rocky Mountain Power Company, of Butte, Mont.: A project on Flathead River below Flathead Lake with five dams and a proposed installed capacity of 272 000 h.p. Action indefinitely suspended awaiting completion of investigation of Columbia Basin irrigation project.

Application for license by the Washington Irrigation and Development Company, of New York, N. Y.: A project on the Columbia River at Priest Rapids with a proposed installed capacity of 300 000 h.p. License authorized March 2, 1925.

Application for preliminary permit by Hugh L. Cooper, M. Am. Soc. C. E., of New York, N. Y.: A project at Z Canyon, Clark Fork, with a proposed installed capacity of 350 000 h.p. Action indefinitely suspended awaiting completion of investigation of Columbia Basin irrigation project.

Application for preliminary permit by the Washington Water Power Company, of Spokane, Wash.: A project on the Columbia River at Kettle Falls with a proposed installed capacity of 153 400 h.p. Permit issued August 26, 1922.

Application for preliminary permit by Mr. Hugh L. Cooper, New York, N. Y.: A project on the Columbia River at Grand Coulee with a proposed installed capacity of 800 000 h.p. Action indefinitely suspended awaiting completion of investigation of Columbia Basin irrigation project.

Trinity River

A Board consisting of W. F. McClure, State Engineer of California, D. C. Henny, M. Am. Soc. C. E., Consulting Engineer, U. S. Bureau of Reclamation, U. S. Grant, 3d, Major, Corps of Engineers, U. S. A., Assoc. M. Am. Soc. C. E., and E. W. Kramer, Hydraulic Engineer, U. S. Forest Service, was appointed by the Federal Power Commission to study the Trinity River (see map, Fig. 10), in California, in order to indicate the most beneficial uses which can be made of its waters and especially to determine whether diversion to the Sacramento River will be to the general benefit. The Board submitted its report in February, 1924. The report will probably not be printed.

The following are the main facts, conclusions, and recommendations set forth in the report:

Trinity River is the main tributary of the Klamath River and drains 2 925 sq. miles of mountainous and wooded area on the west slope of the Coast Range in Northern California. The annual precipitation, dependent on altitude and nearness to the ocean, ranges from 35 to 80 in. Its average for the water-shed is about 50 in. The only town of consequence on the water-shed is Weaverville, an old mining town which derived some importance from hydraulic mining operations that have now practically ceased. Agricultural development is small and is confined mostly to the Hoopa Valley Indian Reservation at the lower end of the stream. There are not more than 6 000

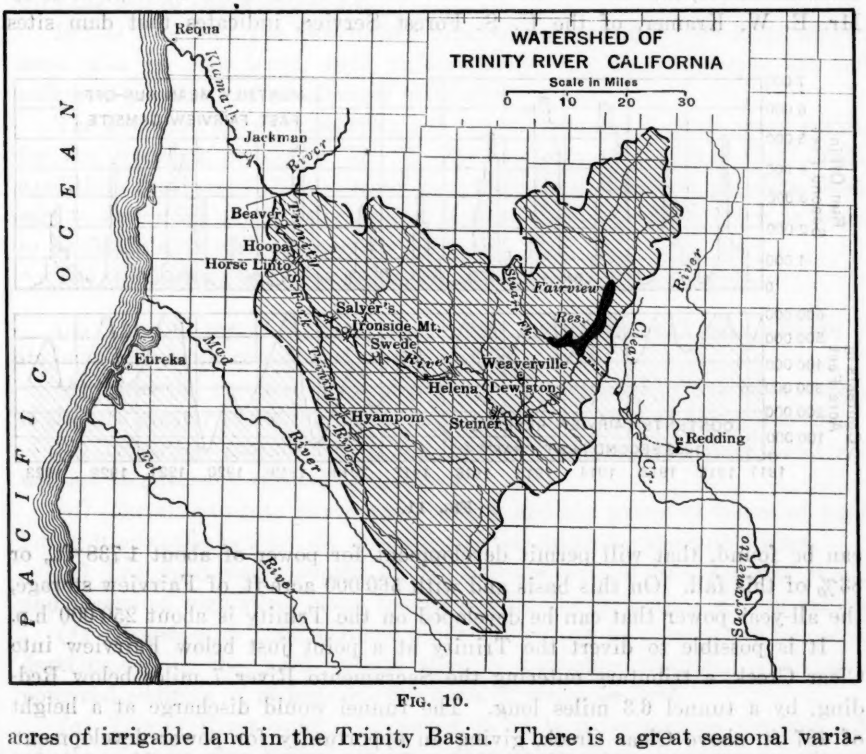


FIG. 10.

acres of irrigable land in the Trinity Basin. There is a great seasonal variation in the flow of the river. At Lewiston, Calif., the mean flow is about 1 600 sec-ft., the maximum is 25 000 sec-ft., and the minimum 52 sec-ft. The areas available for irrigation are so small that the minimum flow is more than enough to irrigate them. It is evident, however, that for successful power development large storage is essential.

There are reservoir sites on Stuart Creek, which enters the Trinity six miles above Lewiston, that have sufficient capacity to regulate the flow of that creek. On the main stream there are only two opportunities for large storage above Lewiston, one at Trinity Center and one at Fairview. The latter is

probably the more favorable, because it takes in the flow of Stuart Creek. It can be developed by a dam 270 ft. high to a capacity of 600 000 acre-ft., is at an elevation of more than 2 000 ft., and receives the flow from the best yielding part of the drainage area.

Fig. 11 shows the run-off of Trinity River past Fairview Dam site and the contents of Fairview Reservoir on the basis of maintaining a regulated flow of 1 053 sec-ft. The suitability of foundations and abutments for a dam 270 ft. in height above the stream bed at this site has not been established by borings.

The average elevation of the surface of Fairview Reservoir would be 2 138 ft. above sea level. From this point to tail-water at the Jackman site, on Klamath River, the fall is 2 091 ft. A reconnaissance of the Trinity, made by Mr. E. W. Kramer, of the U. S. Forest Service, indicates that dam sites

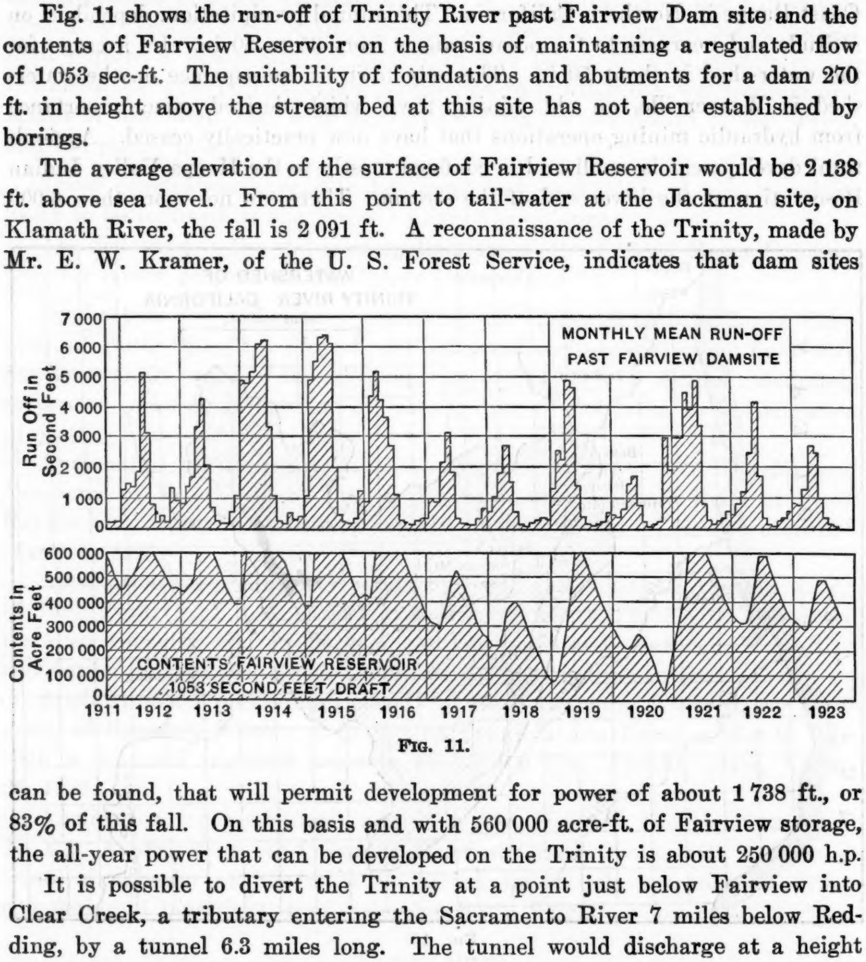


FIG. 11.

can be found, that will permit development for power of about 1 738 ft., or 83% of this fall. On this basis and with 560 000 acre-ft. of Fairview storage, the all-year power that can be developed on the Trinity is about 250 000 h.p.

It is possible to divert the Trinity at a point just below Fairview into Clear Creek, a tributary entering the Sacramento River 7 miles below Redding, by a tunnel 6.3 miles long. The tunnel would discharge at a height of 487 ft. above Clear Creek, giving an opportunity for power development. The tail-water from the power-house would flow into a reservoir on Clear Creek with a dam, at Whiskeytown, 210 ft. high and a storage capacity of 135 000 acre-ft. From this reservoir a conduit and pressure tunnel could conduct the water to a power-house with a head of about 660 ft. on the Sacramento River 3 miles above Redding. The usable head in case of diversion is about 1 268 ft. as against 1 738 ft. on the Trinity, and the all-year power in case of diversion would be about 115 000 h.p. as against 250 000 h.p. on the Trinity. After diversion, it would still be possible to develop about 105 000 all-year horse-power on the Trinity. The net loss of power by diversion is,

therefore, about 30 000 h.p. This loss would be reduced by about 7 000 h.p. if the Iron Canyon project on the Sacramento River were built.

Irrigation on the Trinity will not be injured by diversion of Trinity water at Fairview, whereas about 160 000 acres can be irrigated in the Sacramento Valley by such diversion. It is well recognized that the water supply in the Central Valley of California is sufficient to supply only a part of the irrigable land in the Valley. The shortage is in the San Joaquin or southern half of the Valley, but any water supplied to the west side of Sacramento Valley can be made to release water from the Sacramento to supply the northern part of the San Joaquin. In other words, water diverted from the Trinity will increase the irrigated area in the Central Valley by about 160 000 acres, but with the loss of about 30 000 h.p.

Before a decision can be definitely reached the cost of the diversion project must also be considered, and, unfortunately, the data available are not sufficient to permit of satisfactory estimates.

The Federal Power Commission has recently granted a preliminary permit for the diversion project to Mr. W. H. Sampson, of Corning, Calif., with conditions that require him to make investigations and procure the data necessary to determine whether the project is economically feasible. If it is found to be feasible, it will undoubtedly be built.

The conclusions of the Board were, as follows:

(*a*) The full use of Trinity River water makes essential large storage at its head-waters where required storage facilities exist.

(*b*) Potential power of Trinity River water regulated by available storage is slightly greater if flowing down its natural course than if partly diverted.

(*c*) Irrigation possibilities in the Trinity Basin are relatively small and will not be adversely affected by diversion.

(*d*) The Sacramento River water supply and the potential means of regulating it are sufficient for the irrigation of the entire Sacramento Valley.

(*e*) The joint water supply of the Sacramento and San Joaquin Rivers and means of regulating it are insufficient for the irrigation of both valleys.

(*f*) Diversion of Trinity River water will permit more complete irrigation development in the joint valleys than is otherwise possible, involving a potential addition of at least 160 000 acres of irrigated land.

(*g*) Trinity River water, if diverted, is likely to be used on the west side of the Sacramento Valley, thereby permitting later Sacramento storage development to supply part of the Lower San Joaquin Valley.

(*h*) The only important industrial use made of Trinity River water is in connection with gold dredging. Diversion will not interfere with this industry. Storage above diversion may flood prospective dredging lands in the reservoir site, which fact does not justify delay in reservoir construction.

(*i*) Navigation is confined to Klamath River below the mouth of the Trinity. It may be interfered with to a small extent by diversion which, on the other hand, may benefit Sacramento navigation.

(*j*) The advantages of diversion greatly outweigh its disadvantages.

In harmony with the conclusions, the Board made recommendations as follows:

(*a*) That no power rights be granted to prospective Trinity River water users, that will interfere with ultimate diversion of Trinity water to the Sacramento Valley.

(*b*) That any permission to divert water from the Trinity River to the Sacramento Valley provide for the maintenance of a flow of at least 20 sec-ft. in the Trinity River below the point of diversion.

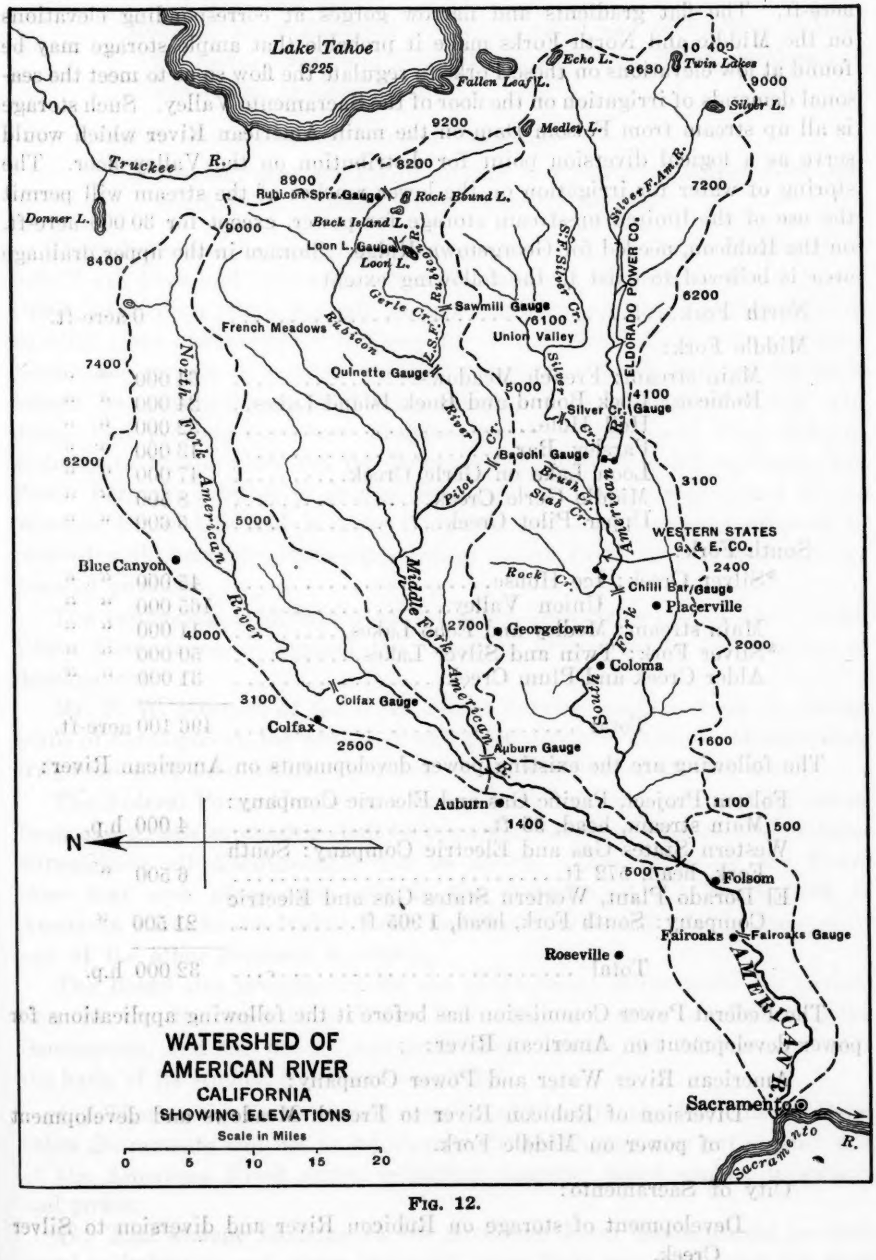
American River

A Board consisting of D. C. Henny, M. Am. Soc. C. E., Consulting Engineer, U. S. Bureau of Reclamation, W. F. McClure, State Engineer of California (represented by Paul Bailey, Assoc. M. Am. Soc. C. E.), U. S. Grant, 3d, Major, Corps of Engineers, U. S. A., Assoc. M. Am. Soc. C. E., and E. W. Kramer, Hydraulic Engineer, U. S. Forest Service, was appointed by the Federal Power Commission to study the American River, in California (see map, Fig. 12), in order to indicate the most beneficial uses that can be made of its waters and especially to determine whether diversion from one to another tributary will be to the general public benefit. The Board submitted its report in April, 1924. The following is abstracted from the report and the records of the Federal Power Commission:

The American River System with its three main Forks—South, Middle, and North—drains the mountainous area lying to the east and northeast of the City of Sacramento. The steep river gradients descending about 6 000 ft. in the upper 50 miles of the stream courses favor economical power development. This is offset to some extent, however, by the wide fluctuations of stream flow throughout the year, by the occurrence at intervals of unusually dry years, and by the paucity of storage facilities at high elevations.

Some agricultural land is found on the ridges between the branches of the rivers at elevations less than 3 500 ft., but the only tract that is feasible of irrigation is on Georgetown Ridge between South and Middle Forks. This tract contains 25 000 acres of irrigable land. The only practicable source of supply for this land is Rubicon Creek, and 50 000 acre-ft. will be required, about 30 000 acre-ft. of which must be supplied from storage. It will be possible to irrigate about 400 000 acres adjacent to the American River and below Folsom on the floor of Sacramento Valley with water from the American River, provided storage can be developed to the amount that appears practicable.

The flood period in the American River generally extends from February to June and during this period about 80% of the annual run-off is discharged. The dry seasonal flow becomes very small in August, September, and October. At times, the September outflow from the entire basin averages as low as 250 sec-ft. The average annual run-off of the American River since 1905 has been more than 3 000 000 acre-ft., but in dry years it has been as low as 1 250 000 acre-ft.



Lake Tahoe
6225
Fallen Leaf L.
Echo L.
Twin Lakes
10 400
9630
9000
Silver L.
Medley L.
9200
8200
Truckee R.
8900
Rubicon Sprs. Gauge
Rock Bound L.
Buck Island L.
Loon L. Gauge
Loon L.
Donner L.
9000
8400
7200
Silver F. Am. R.
S.F. Silver Cr.
ELDORADO POWER CO.
Rubicon R.
Gerle Cr.
L.S.F.
Sawmill Gauge
6100
Union Valley
6200
French Meadows
Rubicon
North Fork American River
7400
Quinette Gauge
5000
Silver Cr.
4100
Silver Cr. Gauge
6200
Bacchi Gauge
Pilot Cr.
Brush Cr.
Slab Cr.
3100
5000
WESTERN STATES G. & E. CO.
Blue Canyon
Middle Fork American
Rock Cr.
2400
Chili Bar Gauge
Placerville
South Fork American
2700
Georgetown
4000
2000
Coloma
3100
Colfax Gauge
Colfax
2500
1600
Auburn Gauge
Auburn
1400
1100
500
N
600
Folsom
Fairoaks
Fairoaks Gauge
Roseville
AMERICAN R.
WATERSHED OF
AMERICAN RIVER
CALIFORNIA
SHOWING ELEVATIONS
Scale in Miles
0
5
10
15
20
Sacramento
Sacramento R.

FIG. 12.

The Coloma Reservoir site at a river elevation of about 600 ft. on the main stream of the South Fork, has a possible storage capacity of at least 300 000 acre-ft. The flat gradients and narrow gorges at corresponding elevations on the Middle and North Forks make it probable that ample storage may be found at low elevations on these Forks to regulate the flow so as to meet the seasonal demands of irrigation on the floor of the Sacramento Valley. Such storage is all up stream from Folsom Dam on the main American River which would serve as a logical diversion point for distribution on the Valley floor. The storing of water for irrigation on the lower reaches of the stream will permit the use of the limited up-stream storage for power, except for 30 000 acre-ft. on the Rubicon, needed for Georgetown Ridge. Storage in the upper drainage area is believed to exist to the following extent:

North Fork	0 acre-ft.
Middle Fork:	
Main stream: French Meadows	64 000 " "
Rubicon: Rock Bound and Buck Island Lakes	24 000 " "
Hell Hole	32 000 " "
Parsley Bar	13 000 " "
Loon Lake on Gerle Creek	47 000 " "
Middle Gerle Creek	8 500 " "
Upper Pilot Creek	5 600 " "
South Fork:	
*Silver Creek: Ice House	45 000 " "
Union Valley	165 000 " "
Main stream: Medley and Echo Lakes	11 000 " "
*Silver Fork: Twin and Silver Lakes	50 000 " "
Alder Creek and Plum Creek	31 000 " "
Total	496 100 acre-ft.

The following are the existing power developments on American River:

Folsom Project, Pacific Gas and Electric Company: Main stream, head, 55 ft.	4 000 h.p.
Western States Gas and Electric Company: South Fork, head, 572 ft.	6 500 "
El Dorado Plant, Western States Gas and Electric Company: South Fork, head, 1 905 ft.	21 500 "
Total	32 000 h.p.

The Federal Power Commission has before it the following applications for power development on American River:

American River Water and Power Company:

Diversion of Rubicon River to French Meadows and development of power on Middle Fork.

City of Sacramento:

Development of storage on Rubicon River and diversion to Silver Creek.

Development of storage and power on Silver Creek.

* Silver Creek must not be confused with Silver Fork.

Development of power on South Fork below Silver Creek and diversion of water from Coloma to Sacramento for domestic and irrigation supply.

Rubicon River diversion was disapproved and preliminary permit for Silver Creek and South Fork, American River, development was issued October 25, 1924.

Western States Gas and Electric Company:

Additional development of storage and power on Silver Fork and South Fork. License issued February 23, 1922.

There have also been applications, now canceled, to divert Middle Fork into North Fork and North Fork into Middle Fork. Middle Fork and Silver Creek have more than enough storage capacity on their head-waters to equalize their stream flow. Rubicon River and South Fork have sufficient head-water storage to regulate their flow in part. North Fork, so far as is known, has practically no head-water storage. It is on account of this condition that projects have been proposed for diverting North Fork and the Rubicon to Middle Fork and for diverting the Rubicon to Silver Creek. On South Fork the head-water storage is controlled by water rights held by the Western States Gas and Electric Company, and that Company proposes to proceed with complete development above Silver Creek as fast as its power demand permits.

The conflicting applications on the other tributaries caused the Federal Power Commission to appoint the Board to determine the best scheme of development.

Mr. E. W. Kramer, of the U. S. Forest Service, made a study of various plans of development, the results of which are given in Table 6. Credits from irrigation have been arbitrarily based on an estimate of $20 per acre.

The Federal Power Commission has adopted a general rule, that diversions from one stream to another shall be permitted only when positive advantages outweighing all disadvantages can be shown. The studies of the Board show that such advantages exist in the diversion of the North Fork of American River to the Middle Fork, and that they do not exist in the case of any of the other proposed diversions.

The Board also investigated the use of American River water for navigation in the Sacramento, and for improving salt-water conditions below Sacramento, and the use of American River storage for flood control. On the basis of its studies, the Board concluded:

(*a*) That navigation and salt-water conditions in the Sacramento River below Sacramento will not be injuriously affected by any now anticipated uses of the American River water, including domestic water supply, irrigation, and power.

(*b*) That storage facilities in the American River Basin should be dedicated to irrigation and power primarily, since their economic value for these purposes is too great to justify their developments solely for flood control.

(*c*) That the fullest practicable irrigation of the cultivable lands of the American River Basin is in the interest of the public and requires that any power or irrigation projects developing upper storage reservoirs in the Rubicon drainage area under Federal Power license, shall be obligated to supply at reservoirs, shortage in irrigation water for approximately 25 000 acres on the Georgetown Ridge, at a price based on investment and cost of service, and not on value of power lost by supplying this irrigation need.

TABLE 6.

Plan of development.	All-year horse-power.	Total cost, exclusive of trans-mission.	Credit for Irrigation.		Net cost of power.	Cost per all-year horse-power.
			Acres.	Amount.		
1.—Diversion North Fork to Middle Fork:						
Diversion Upper Rubicon to Silver Creek:						
North and Middle Forks......	80 000	$20 120 000	36 000	$ 720 000	$19 400 000	$242
Rubicon..........	37 000	7 960 000	26 000	520 000	7 440 000	201
Silver Creek..........	136 000	32 099 000	87 000	1 740 000	30 359 000	223
Total..........	253 000	$60 179 000	149 000	$2 980 000	$57 199 000	$226
2.—Diversion North Fork to Middle Fork:						
North and Middle Forks......	87 000	$21 250 000	36 000	$ 720 000	$20 530 000	$236
Rubicon..........	99 000	20 509 000	77 000	1 540 000	18 969 000	192
Silver Creek..........	79 000	19 250 000	45 000	900 000	18 350 000	232
Total..........	265 000	$61 009 000	158 000	$3 160 000	$57 849 000	$218
3.—No Diversion:						
North Fork..........	0					
Middle Fork..........	45 000					
Rubicon..........	99 000			Not computed		
Silver Creek..........	79 000					
Total..........	223 000	....	......	....	....	....
4.—Diversion North Fork to Middle Fork:						
Diversion Upper Rubicon to Middle Fork:						
Middle Fork..........	118 000					
Rubicon..........	0			Not computed		
Silver Creek..........	79 000					
Total..........	197 000	....	......	....	....	....

(*d*) That until investigations show that large storage for Valley irrigation cannot be feasibly developed on the lower reaches of the North and Middle Forks below River Elevation 1 150, it is inadvisable to permit power development which would interfere with irrigation storage below this elevation.

(*e*) That the Coloma Reservoir has sufficient capacity and is located so that it can regulate for the benefit of irrigation, almost the entire flow of the South Fork of the American River below power developments. Its primary value is for irrigation storage.

(*f*) That the Folsom dam site admits raising the dam to a considerable additional height and that this site is located at the logical point for diverting American River water for all lower gravity irrigation.

(*g*) That the potential power of the North and Middle Forks of the American River can be best developed by the diversion of the Upper North Fork drainage to French Meadows Reservoir on the Middle Fork.

(*h*) That the diversion of Rubicon water to the Middle Fork would be wasteful of power.

(*i*) That the diversion of Rubicon water to Silver Creek would prevent maximum power utilization and would tend to increase the total power cost.

(*j*) That the municipal power, domestic water supply, and irrigation requirements of the City of Sacramento and the Sacramento Municipal Utility District, for a reasonably extended future, can be satisfactorily supplied from Silver Creek without Rubicon diversion.

Stanislaus River

The Federal Power Commission also requested the Board which reported on the Trinity and American Rivers in California to report on a proposed enlargement of the present development on the North Fork of Stanislaus River, California, with a view to determining the best method of development. The report was submitted December 1, 1924, and will probably not be printed. The questions involved are probably not of sufficient general interest to merit discussion herein.

Kings River

The San Joaquin Light and Power Corporation has been planning to undertake the complete development for power of North Fork, Kings River (see plan, Fig. 13), since 1913. After many delays, on July 28, 1922, the Corporation was issued a Federal Power Commission license for the entire project. Section 9 (*b*) of the Federal Water Power Act, reads, as follows:

"Satisfactory evidence that the applicant has complied with the requirements of the laws of the State or States within which the proposed project is to be located with respect to bed and banks and to the appropriation, diversion, and use of water for power purposes and with respect to the right to engage in the business of developing, transmitting, and distributing power, and in any other business necessary to effect the purposes of a license under this act."

On account of this section, however, the license was made subject to the following proviso:

"Provided, that this license shall not become effective with respect to the following separable parts of the above described project works until the Licensee has complied with the requirements of the laws of the State of California with respect to appropriation, diversion, and storage of water contemplated in the development of said separable parts:

(*a*) Wishon Dam, Reservoirs and Power Plant
(*b*) Meyer Dam and Reservoir
(*c*) Peart Dam, Reservoir and Power Plant
(*d*) Rancheria Creek Diversion Dam and Tunnel
(*e*) Bear Creek Diversion Dam and Canal."

The Corporation was issued water permits by the State for the use of the natural flow of North Fork, Kings River, but its applications to store water were held up on account of protests from the irrigation interests that use Kings River water. The Division of Water Rights, State Department of Public Works of California, notified the Commission, however, that it was expected that water rights for the entire project of the San Joaquin Light and Power Corporation would be granted as soon as action could be taken on the Pine Flats Reservoir project, application for which was made to the State on January 1, 1916.

The Pine Flats project proposes the construction of a reservoir with a capacity of 600 000 acre-ft., located at the outlet of Kings River Canyon, just above Piedra, Calif. The primary object of the project is to regulate the flow of Kings River for irrigation. The area irrigated at present varies from 500 000 to 700 000 acres, and it is estimated that about 1 000 000 acres can be irrigated when the Pine Flats project is completed. With the possible exception of Kern River, Kings River presents more complications for the organization of a comprehensive irrigation development than any other stream in California. The are twenty-three different groups of interests, with ten or twelve additional subsidiary interests using Kings River water. The water rights are very complicated and, as might be expected, are more or less in conflict. The interest having the last water right can take no water except when the flow in the river exceeds 9 500 cu. ft. per sec. The Kings River territory is one of the largest and most successful irrigation developments in the United States. The present use of water is not as economical as it should be, and the advantages of expanding the development by construction of the Pine Flats project are so great that undoubtedly the difficulties will eventually be overcome.

The low-water flow of North Fork, Kings River, is so meager and the cost of power development is so great that the power project of the San Joaquin Light and Power Corporation is not economically feasible without storage. The complete development will include four reservoirs, as shown in Table 7.

The power capacity of the project under the Federal Power Commission's definition will be about 215 000 h.p., and the proposed installed capacity about 400 000 h.p.

There are also considerable power possibilities on the South and Middle Forks of Kings River, but the region is very rugged and difficult of access. and the projects will be relatively expensive so that some years may elapse before they are built. After the construction of the Pine Flats project, the development of power on South and Middle Forks will benefit irrigation development, but the data available are so meager that the ultimate relation between power on these Forks and irrigation cannot now be determined.

Since 1917 the diversion of water from Kings River for irrigation has been supervised by a Watermaster appointed by the State Division of Water Rights. Table 8 has been compiled from the records of the Watermaster. Table 9 has

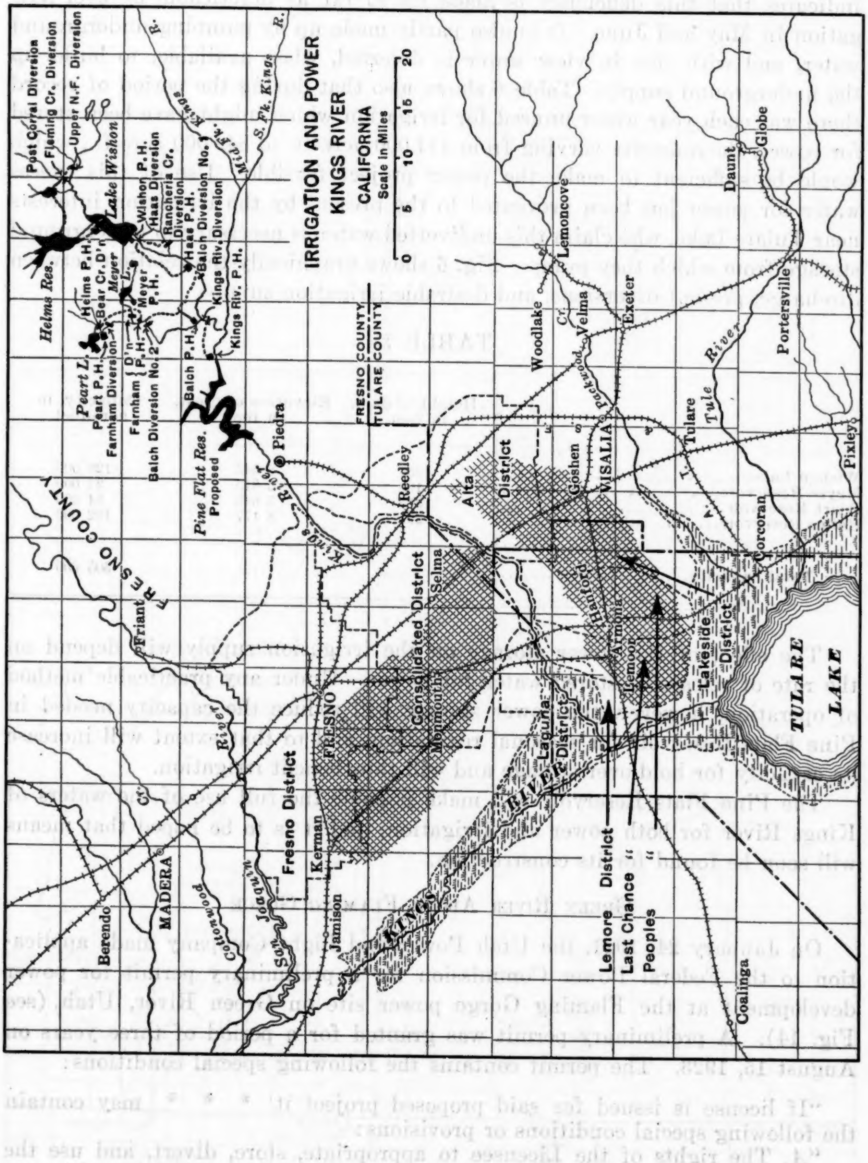

IRRIGATION AND POWER
KINGS RIVER
CALIFORNIA
Scale in Miles
0 5 10 15 20
Post Corral Diversion
Fleming Cr. Diversion
Upper N.F. Diversion
Lake Wishon
Wishon P.H.
Haas Diversion
Rancheria Cr. Diversion
Haas P.H.
Balch Diversion No.1
Kings Riv. Diversion
Kings Riv. P.H.
Helms Res.
Helms P.H.
Bear Cr. D'n
Meyer
Meyer P.H.
Peart L.
Peart P.H.
Farnham Diversion
Farnham P.H.
Balch Diversion No.2
Balch P.H.
Pine Flat Res.
Proposed
Piedra
FRESNO COUNTY
TULARE COUNTY
Friant
MADERA
Berendo
Cottonwood
San Joaquin River
Fresno District
Kerman
FRESNO
Consolidated District
Monmouth
Selma
Reedley
Alta
District
Jamison
KINGS
RIVER
Laguna
District
Hanford
Armona
Lemoore
Last Chance
Peoples
District
Goshen
VISALIA
Woodlake
Lemoncove
Velma
Exeter
Tulare
Tule River
Porterville
Globe
Corcoran
Lakeside District
TULARE LAKE
Pixley
Coalinga

FIG. 13.

been compiled from the discharge records of the Watermaster and the irrigation demands set forth in *Bulletin No. 6,* "Irrigation Requirements of California Lands." Table 9 shows that the unregulated discharge is deficient in July, August, September, and October properly to supply the irrigation need, the deficiency running from 318 000 acre-ft. to 660 000 acre-ft. Table 8 indicates that this deficiency is made up so far as practicable by over-irrigation in May and June. It is also partly made up by pumping underground water, and with this in view water is diverted, when available, to build up the underground supply. Table 8 shows also that during the period of record there was each year water unused for irrigation which might have been stored for power, the quantity varying from 111 000 acre-ft. to 842 000 acre-ft., which would be sufficient to make the power project feasible. Use of this excess water for power has been prevented to the present by the irrigation interests near Tulare Lake, who claim this undiverted water is necessary to underground storage from which they pump. Fig. 6 shows graphically the relation between discharge, present diversions, and desirable irrigation supply.

TABLE 7.

	Height of dam, in feet.	Elevation of crest, in feet.	Capacity, in acre-feet.
Wishon Lake	250	6 545	128 000
Meyer Reservoir	190	6 845	21 000
Peart Reservoir	315	5 645	54 600
Helms Reservoir	297	8 177	102 000
Total capacity	....	....	305 600

The effect of the power storage on the irrigation supply will depend on the rate of release of stored water for power. Under any practicable method of operation, however, the power storage will reduce the capacity needed in Pine Flats Reservoir for seasonal regulation, and to that extent will increase its capacity for hold-over storage and will thus benefit irrigation.

The Pine Flats Reservoir will make possible the full use of the waters of Kings River for both power and irrigation, and it is to be hoped that means will soon be found for its construction.

Green River Above Flaming Gorge

On January 24, 1921, the Utah Power and Light Company made application to the Federal Power Commission for a preliminary permit for power development at the Flaming Gorge power site on Green River, Utah (see Fig. 14). A preliminary permit was granted for a period of three years on August 15, 1923. The permit contains the following special conditions:

"If license is issued for said proposed project it * * * may contain the following special conditions or provisions:

"*A.* The rights of the Licensee to appropriate, store, divert, and use the waters of said Green River in connection with said project shall be subject to and be limited by the provisions of the 'Colorado River Compact' signed at

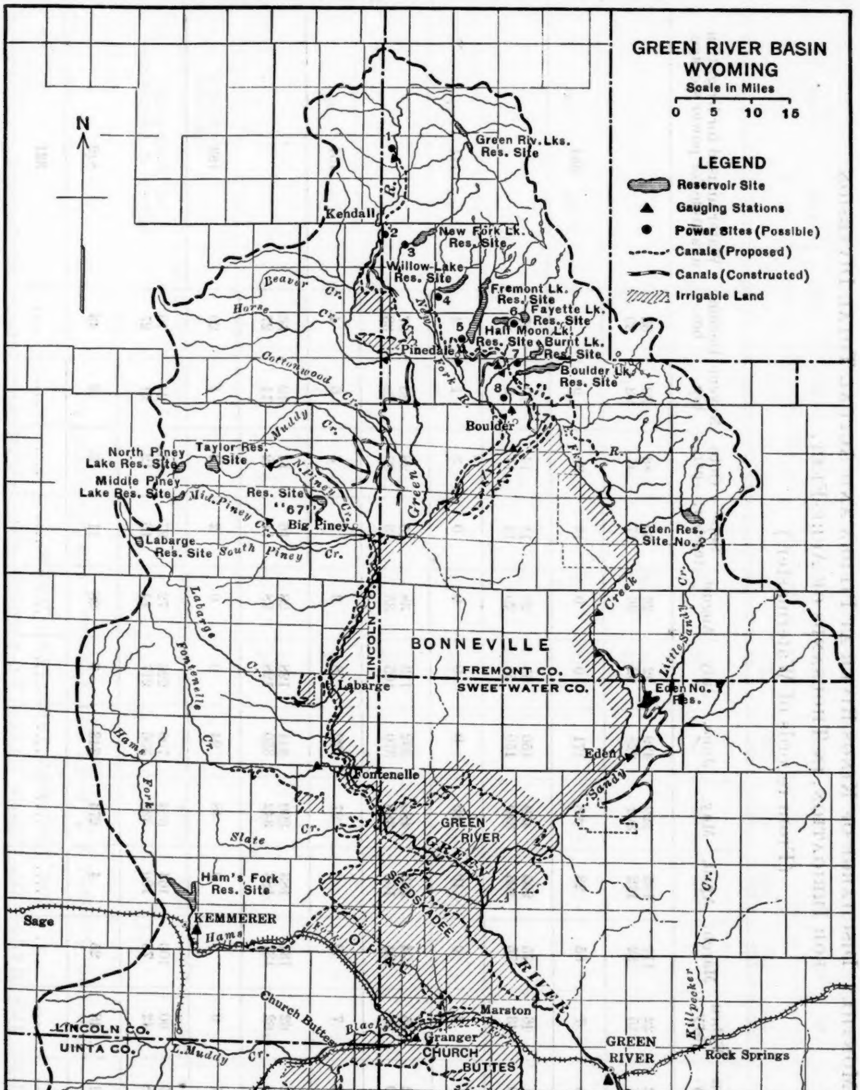

GREEN RIVER BASIN
WYOMING
Scale in Miles
0 5 10 15
LEGEND
Reservoir Site
Gauging Stations
Power Sites (Possible)
Canals (Proposed)
Canals (Constructed)
Irrigable Land
N
Green Riv. Lks. Res. Site
Kendall
New Fork Lk. Res. Site
Willow Lake Res. Site
Fremont Lk. Res. Site
Fayette Lk. Res. Site
Half Moon Lk. Res. Site
Burnt Lk. Res. Site
Pinedale
Boulder Lk. Res. Site
Boulder
Beaver Cr.
Horse Cr.
Cottonwood Cr.
Muddy Cr.
New Fork R.
North Piney Lake Res. Site
Taylor Res. Site
Middle Piney Lake Res. Site
Mid. Piney Cr.
N. Piney Cr.
Res. Site "67"
Big Piney
Labarge Res. Site
South Piney Cr.
Green R.
Eden Res. Site No. 2
LINCOLN CO.
BONNEVILLE
FREMONT CO.
SWEETWATER CO.
Labarge
Labarge Cr.
Fontenelle Cr.
Little Sandy Cr.
Eden No. 1 Res.
Eden
Sandy
Hams Fork
Fontenelle
Slate Cr.
GREEN RIVER
SEEDSKADEE
Ham's Fork Res. Site
Sage
KEMMERER
Hams Fork
Church Buttes
Blacks Fork
Marston
Granger
CHURCH BUTTES
GREEN RIVER
Killpecker Cr.
Rock Springs
LINCOLN CO.
UINTA CO.
L. Muddy Cr.
Carter
Uinta No. 3 Res. Site
Patterson Lake Res. Site
Ft. Bridger
Lyman
EVANSTON
Muddy
Flaming Gorge Reservoir Site
Cottonwood
Smith Fork
Blacks Fork
WYOMING
Henrys Fork
Res. Sites
No. 3
No. 2
No. 1
Basin Res. Site
Beaver
Burnt
UTAH

FIG. 14.

TABLE 8.—MONTHLY DISCHARGE OF KINGS RIVER AT PIEDRA AND ACTUAL TOTAL DIVERSION FOR IRRIGATION, IN THOUSANDS OF ACRE-FEET.

(From records of Watermaster.)

Year.		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	Water unused for irrigation available for power storage.
1918	Discharge	12	22	117	195	329	499	92	26	30	86	32	35	
	Diversion	7	15	52	172	301	328	92	26	17	47	23	30	
	Water unused	5	7	65	23	28	171	0	0	13	39	9	4	364
1919	Discharge	31	46	75	205	459	150	55	20	11	14	10	27	
	Diversion	30	43	75	195	370	150	55	20	11	11	9	23	
	Water unused	1	3	0	10	89	0	0	0	0	3	1	4	111
1920	Discharge	22	35	104	163	485	392	113	38	15	28	35	37	
	Diversion	20	28	104	163	388	370	105	35	13	25	29	33	
	Water unused	2	7	0	0	97	22	8	3	2	3	6	4	154
1921	Discharge	52	62	131	183	390	444	128	28	16	14	12	56	
	Diversion	45	62	131	183	342	350	128	28	14	12	11	27	
	Water unused	7	0	0	0	48	94	0	0	2	2	1	29	183
1922	Discharge	89	90	100	164	664	709	226	72	27	15	24	70	
	Diversion	38	42	75	160	390	356	217	44	16	12	21	27	
	Water unused	51	48	25	4	274	343	9	28	11	3	3	43	842
Average		....	....	....	....	....	....	....	....	....	....	....	....	331

TABLE 9.—MONTHLY DISCHARGE OF KINGS RIVER AT PIEDRA AND DESIRABLE MONTHLY SUPPLY FOR PRESENT IRRIGATED AREA.

(700 000 Acres at 2 Acre-Ft. per Year = 1 400 000 Acre-Ft.)

Year.		January.	February.	March.	April.	May.	June.	July.	August.	September.	October.	November.	December.	TOTALS, IN THOUSANDS OF ACRE-FEET.	
															Storage required.
1918	Discharge..	12	22	117	195	329	499	92	26	30	86	32	35		
	Demand...	....	28	70	154	238	252	252	210	140	56	....	....		
	Difference.	+12	− 6	+47	+ 39	+ 91	+247	−160	−184	−110	+30.	+32	+35	+ 533	−460
1919	Discharge..	31	46	75	205	459	150	55	20	11	14	10	27		
	Demand...	....	28	70	154	238	252	252	210	140	56	....	....		
	Difference.	+31	+18	+ 5	+ 91	+221	−102	−197	−190	−129	−42	+10	+27	+ 403	−660
1920	Discharge..	22	35	104	163	485	392	113	38	15	28	35	37		
	Demand...	....	28	70	154	238	252	252	210	140	56	....	....		
	Difference.	+22	+ 7	+34	+ 9	+247	+140	−139	−172	−125	−28	+35	+37	+ 531	−464
1921	Discharge..	52	62	131	183	390	444	128	28	16	14	12	56		
	Demand...	....	28	70	154	238	252	252	210	140	56	....	....		
	Difference.	+52	+34	+61	+ 29	+152	+192	−124	−182	−124	−42	+12	+56	+ 588	−472
1922	Discharge..	89	90	100	164	664	709	226	72	27	15	24	70		
	Demand...	....	28	70	154	238	252	252	210	140	56	....	....		
	Difference.	+89	+62	+30	+ 10	+426	+457	− 26	−138	−113	−41	+24	+70	+1118	−318

Santa Fé, New Mexico, on November 24, 1922, if, when, and as said Compact is ratified.

"*B.* Said rights to appropriate, store, divert, and use waters of Green River shall be subordinate to all present and future rights to the use of the waters of said river above said dam for purposes of irrigation or of domestic water supply."

The second of these special conditions was inserted by the Federal Power Commission to meet objections by the State of Wyoming which, with the other arid States, is following a policy of giving the use of water for irrigation unqualified precedence over its use for power.

The Utah Power and Light Company has completed its investigation of the power site and prepared plans for the project structures, but has not yet applied for a license. It is understood that the Company feels that it can not finance the project if Special Condition *B* is included without qualification in the license, and therefore it has modified its construction program and will extend its development on Bear River to meet immediate market demands in the hope that sentiment in Wyoming will change so that the reservation of water for irrigation use on Green River may be definitely limited to what will be needed to meet the probable growth of irrigation. A brief review of the physical features of the situation will show the principal factors involved.

The power project proposed consists of a dam about 220 ft. high in Horse Shoe Canyon, about 40 miles by air line below Green River, Wyo.; a tunnel, 3½ miles long, leading from the reservoir above the dam through Bear Mountain; and a power-house on the river, 15 miles by river below the dam. The latter part of the project may be modified on further study, but the power capacity will not materially change. The reservoir capacity is 3 476 000 acre-ft., but probably only the top 50 ft., with a capacity of 1 710 000 acre-ft., will be drawn upon as that amount of storage is sufficient to equalize the flow of the stream. The average power capacity of the project continuously available will be about 75 000 h.p. and the installed capacity will be more than 100 000 h.p. There appear to be no serious construction difficulties, and although the site is about fifty miles from the nearest railroad and the project is not a cheap one, it probably can be built at a cost that will not raise present power rates. The Company now furnishes power to most of the State of Utah, and this project would be connected to and supplement its power system.

There is considerable difference of opinion as to the amount of irrigation development that is feasible in Green River Valley, Wyoming. Some believe that there will be little increase over the present while others believe that the entire stream flow above Green River, Wyo., will be needed for irrigation. The following information is taken mostly from unpublished reports by Mr. R. R. Wooley, of the U. S. Geological Survey, dated February, 1923, and by the U. S. Bureau of Reclamation, dated February, 1924:

The Green River Basin in Wyoming covers an area of about 15 000 sq. miles. All the irrigable land lies at an elevation of 6 000 ft. or more above sea level. The average precipitation in the valley ranges from 6 in. in the south to 11 in. in the north with much higher precipitation on the mountains. The average annual temperature varies from about 42° Fahr. at Green River,

Wyo., to 34° Fahr. in the northern part of the valley. The growing season varies from 105 days to 30 days. Frosts may be expected any month.

TABLE 10.—FROST DATA.

Station.	Number years of record.	Average date of first killing frost, fall.	Average date of last killing frost, spring.	Earliest date of killing frost, fall.	Latest date of killing frost, spring.
Green River.......	10	September 4	June 10	August 11	June 23
Daniel..............	13	August 18	June 22	July 7	July 9
Pinedale...........	7	August 17	June 29	July 31	July 16
Manila..............	8	September 8	June 22	August 25	July 10

Coal mining and stock-raising are the principal occupations. The population is about 20 000, more than 65% of which live in towns along the Union Pacific Railway. The irrigable land lies from 10 to 100 miles from the railroad. The present irrigated area is largely made up of bottom-lands developed by individual effort to supplement the stock industry. The principal crop is wild hay.

Diversion from the tributaries is not difficult, but for the large projects, with lands away from the river, long main canals are required because of light river grades. Under the Carey Act, many projects have been outlined by surveys and much development proposed and several projects have been partly built. A total of about 500 000 acres is covered by permits for appropriation of water. Diversion demands range from 2.5 acre-ft. to 3.0 acre-ft. per year, with a net consumption of water averaging 1.5 acre-ft. Lack of reservoir sites at strategical points will prevent the use of the entire run-off of the basin.

Table 11 gives the U. S. Reclamation Bureau's classification of irrigable lands.

Class A.—

1.—Undeveloped areas of constructed or partly constructed projects not requiring undue expenditures.

2.—New projects concerning which no serious construction difficulties are known and for which water supply seems ample.

3.—Increase in area under small ditches already built or anticipated to be built shortly.

Class B.—

Developments similar to those of Class A but by reason of relatively higher cost not likely to be carried through in the immediate future.

Class C.—

Developments possibly feasible from the standpoints of construction and water supply, but out of the question in the near future by reason of excessive cost.

The average annual discharge of Green River at Flaming Gorge, deducting present irrigation use and probable evaporation losses in the reservoir, is about 1 810 000 acre-ft., giving about 2 500 cu. ft. per sec. available for power.

According to the estimate of the U. S. Bureau of Reclamation the depletion of flow caused by future development per Table 11 is:

Class *A* projects,	279 000 acre-ft. per year equivalent to	385	cu. ft. per. sec.
Class *B* "	, 255 000 " " " " "	352	" " " "
Class *C* "	, 236 000 " " " " "	326	" " " "
Total.....	770 000 " " " " "	1 063	" " " "

TABLE 11.—Classification of Projects.

Project.	Project area, in acres.	Acres irrigated in 1922.	Class A.	Class B.	Class C.
Above Green River City:					
Fontenelle Creek	4 000	....	....	4 000	....
La Barge Project	3 000	....	3 000	....	....
Big Piney-La Barge	6 000	....	6 000	....	....
"67" Reservoir	2 200	....	2 200	....	....
Cottonwood Development Company	40 000	2 000	38 000	....	....
Uinta-Fremont	14 000	1 000	13 000	....	....
Apex Project	4 000	....	4 000	....	....
Fremont Lake	7 000	1 500	5 500	....	....
Boulder	8 600	6 000	2 600	....	....
East Fork Canal	7 800	2 000	5 800	....	....
Willow Lake	1 000	....	1 000	....	....
Paradise	3 500	....	3 500	....	....
Bertram	900	500	400	....	....
Eden	40 000	7 000	33 000	....	....
Small ditches	180 000	120 000	30 000	30 000	....
Green River	60 000	....	....	60 000	....
Seedskadie	93 000	....	....	27 000	66 000
Bonneville	69 000	....	....	....	69 000
Total	544 000	140 000	148 000	121 000	135 000
Hams Fork:					
Opal project	40 000	....	....	40 000	....
Small ditches	17 000	12 000	5 000	....	....
Total	57 000	12 000	5 000	40 000	....
Blacks Fork:					
Lyman ditches	63 000	31 000	32 000	....	....
Small ditches	44 000	29 000	10 000	5 000	....
Total	107 000	60 000	42 000	5 000	....
Henrys Fork:					
Small ditches	20 000	8 000	7 000	5 000	....
Grand total	728 000	220 000	202 000	171 000	135 000

At the Flaming Gorge project 1 cu. ft. of water will produce about 30 h.p., and the average continuous horse-power will be:

With present water supply......	75 000 h.p., or	492 750 000	kw-hr. per year
After Class *A* developments.......	63 450 " "	416 866 000	" " "
" Class *B* "	52 890 " "	347 487 000	" " "
" Class *C* "	43 110 " "	283 233 000	" " "

The estimated cost of the Flaming Gorge project is \$24 300 000. The power at the switchboard will cost about:

With present water supply	\$0.00589	per kw-hr.
After Class *A* developments	0.00697	" " "
" Class *B* "	0.00836	" " "
" Class *C* "	0.01026	" " "

Present power rates in Utah will hardly justify a cost of \$0.006 per kw-hr. for power at this plant. It is possible that as the load grows and more head can be developed below Flaming Gorge the regulated flow through the greater head will reduce the costs so that the project will be feasible with a reduced flow, but, at present, any considerable reduction in flow will make the project impossible.

Any prediction as to rate of growth of irrigated area is perhaps hazardous, but a study of the rate of growth of Green River Valley, Wyoming, is interesting.

Fig. 15 shows the growth of the irrigated area from 1889 to 1922. It also shows what the future growth may be: First, by projection of the curve; and, second, by projection of a straight line through the points for 1909 and 1922. Considering the very short growing season, the inaccessibility of the region, and the expense of developing the larger projects, there is reason to believe that the irrigated area will not increase faster than is indicated by projection of the curve, and it certainly is unlikely that it will increase faster than the straight-line projection. In other words, 50 years hence, the irrigated area may have increased to 320 000 acres, and it is unlikely that it will have increased to 445 000 acres. Successful agriculture in this region of high altitude is likely to be limited in the future, as in the past, to the raising of winter feed for stock that can be grazed on the many miles of adjoining range. Such irrigation is usually confined to individual effort on small ditches. Referring again to the classification of the U. S. Reclamation Bureau, it will be seen that, under Class *A*, there are 52 000 acres that can be irrigated with small ditches.

Irrigation is much better known than it was twenty years ago. Private capital will be difficult to obtain for any large project in Green River Valley, Wyoming. Strong efforts are still made from time to time to get the Federal or the State Governments to undertake projects that are considered infeasible for private interests, but fairly strong opposition generally is met, and lately such undertakings have been very rare.

The probabilities are that irrigation in Green River Valley, Wyoming, will not seriously interfere with the Flaming Gorge power project but, in order to finance that project, it may be necessary to set a schedule specifying the rate at which the water supply may be depleted for irrigation.

Salt River Valley

Since 1912, Frank G. Baum, M. Am. Soc. C. E., of San Francisco, Calif., has had under consideration and, since 1914, has been trying to obtain Federal authority for a power project on Black River, a tributary of Salt River, in Arizona. (See Fig. 16.) In December, 1920, he made application to the Federal Power Commission for a preliminary permit. The project included three

reservoirs with a combined capacity of 35 000 acre-ft. at an elevation about 8 000 ft. above sea level. The power-house would have a head of about 870 ft., and an installed capacity of about 10 000 h.p. It was proposed to find a market for the power in Globe, Clifton, Miami, Jerome, or other mining centers.

The Salt River Valley Water Users' Association made a strong protest against the project on the ground that storage of water on Black River in low-flow years would deplete the water supply for about 200 000 acres of irrigated land in the Salt River project by at least 10 000 acre-ft., and would also cause a power loss of about 700 000 kw-hr. per year which, if available, would pump about 10 000 acre-ft. of ground-water. The total loss of water,

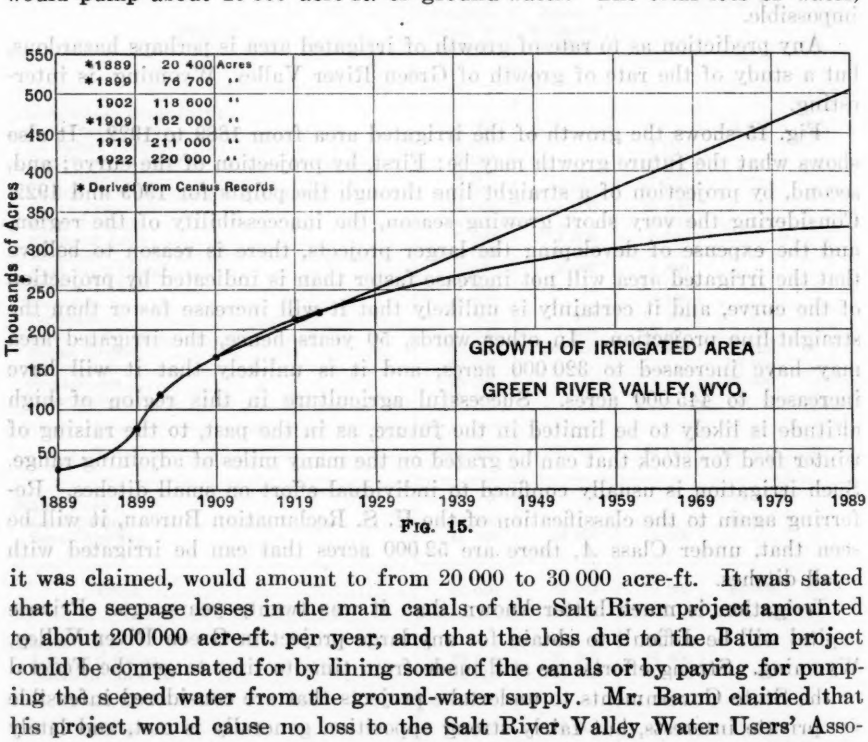


FIG. 15.

it was claimed, would amount to from 20 000 to 30 000 acre-ft. It was stated that the seepage losses in the main canals of the Salt River project amounted to about 200 000 acre-ft. per year, and that the loss due to the Baum project could be compensated for by lining some of the canals or by paying for pumping the seeped water from the ground-water supply. Mr. Baum claimed that his project would cause no loss to the Salt River Valley Water Users' Association, but on the contrary would be beneficial. The data available are too meager to determine accurately what increased evaporation loss might be due to the maintenance of a fairly uniform flow throughout the year in the 125 miles of river channel between the Baum project and Roosevelt Reservoir, instead of sending most of the water down on the flood flows, as at present. From such data as are available studies were made by engineers of the Federal Power Commission. These studies indicate that for about 5 years in the past 35, the Baum project would have diminished the available water supply by perhaps as much as 2 000 acre-ft., out of a total of about 900 000 acre-ft., and would have reduced the mean head at the power plant at Roosevelt Dam by perhaps 2 ft. out of 80 ft. For 17 years it would have increased the water supply and head for power, and, during the other 13 years, its effect would

have been negligible. As the detrimental effect during the 5 years of low flow can be remedied by lining canals or by supplying power for pumping ground-water, the problem is one of determining the cost of such operations and balancing that cost against the value of extra power in the 17 years during which the Salt River project would have benefited. It is doubtful whether the balance would be in favor of, or against, the Baum project, but it is fairly certain that the amount involved would be small.

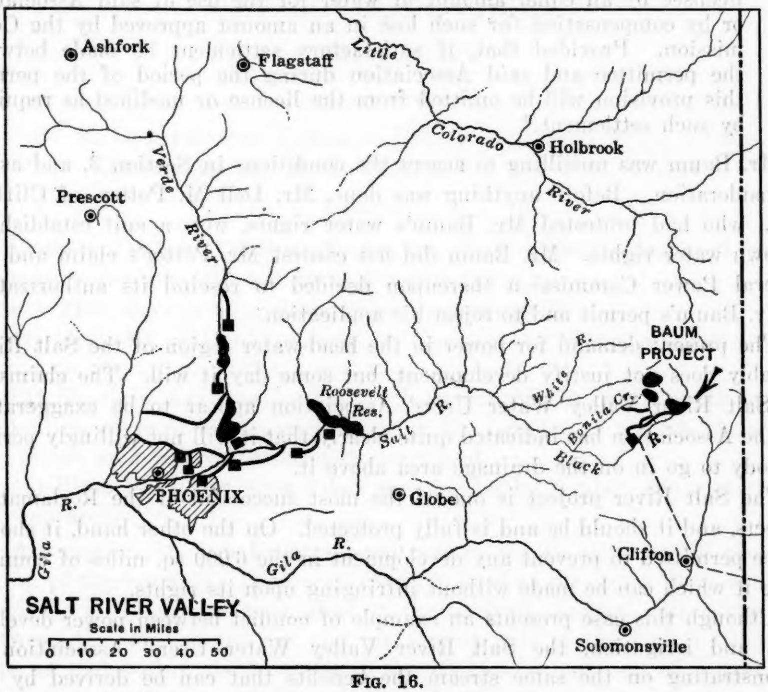


FIG. 16.

The drainage area above Roosevelt Dam contains about 6 000 sq. miles and is capable of economically developing about 75 000 continuous h.p. whenever a market offers, provided head-water storage can be developed. The State of Arizona has adequate laws to protect the existing water rights of the Salt River Valley Water Users' Association, but practically the entire State supported the Association's protest and the Federal Power Commission, wishing to avoid putting the Association to the expense of a lawsuit, decided to attempt to adjudicate the question itself, and, therefore, inserted in the Baum permit the following conditions:

"For the purpose of safeguarding the interests of the United States and the Salt River Valley Water Users' Association, the permittee shall, prior to the issue of a license, amend the notices of appropriation of the waters of the tributaries of Salt River so as to eliminate therefrom all reference to appropriations for irrigation purposes, and the license, if issued, shall contain a condition that the licensee shall not use said waters for the purposes of irrigating lands, and that the licensee shall:

"1. Permit the total normal flow of Black River to pass continuously through the power plant or through by-passes at the power plant,

except when the flow of Salt River at the diversion dam at the head of the Roosevelt Power Canal of the Salt River project is greater than 250 cu. ft. per sec.

"2. Return all waters diverted or stored for any purpose to the natural channel of Black River immediately below the power plant.

"3. Replace to the satisfaction of the Commission any net loss of water to the Salt River Valley Water Users' Association caused by evaporation from the storage reservoirs and other project works of the licensee by an equal amount of water for the use of said Association or by compensation for such loss in an amount approved by the Commission. Provided that, if satisfactory settlement be made between the permittee and said Association during the period of the permit, this provision will be omitted from the license or modified as required by such settlement."

Mr. Baum was unwilling to accept the conditions in Section 3, and asked reconsideration. Before anything was done, Mr. Dell M. Potter, of Clifton, Ariz., who had protested Mr. Baum's water rights, won a suit establishing his own water rights. Mr. Baum did not contest Mr. Potter's claim and the Federal Power Commission thereupon decided to rescind its authorization of Mr. Baum's permit and to reject his application.

The present demand for power in the head-water region of the Salt River probably does not justify development, but some day it will. The claims of the Salt River Valley Water Users' Association appear to be exaggerated, but the Association has indicated quite clearly that it will not willingly permit anybody to go in on the drainage area above it.

The Salt River project is one of the most successful of the Reclamation projects, and it should be and is fully protected. On the other hand, it should not be permitted to prevent any development in the 6 000 sq. miles of country above it which can be made without infringing upon its rights.

Although this case presents an example of conflict between power development and irrigation, the Salt River Valley Water Users' Association is demonstrating on the same stream the benefits that can be derived by co-ordinating irrigation and power. The Association is developing power itself wherever such development is practicable in its district, and the returns from the sale of power are contributing materially toward paying the cost of irrigation.

These cases illustrate fairly well the problems of the Federal Power Commission on the subject of co-ordinating irrigation and power. The function of the Commission in this respect is prescribed by Section 10 (*a*) of the Federal Water Power Act, which reads as follows:

"That the project adopted, including the maps, plans, and specifications, shall be such as in the judgment of the Commission will be best adapted to a comprehensive scheme of improvement and utilization for the purposes of navigation, of water-power development, and of other beneficial public uses; and if necessary in order to secure such scheme the commission shall have authority to require the modification of any project and of the plans and specifications of the project works before approval."

This function has been regarded by the Commission as one of the most important with which it is charged.

THE REINFORCED CONCRETE ARCH IN SEWER CONSTRUCTION: A REVIEW OF PAST PRACTICE IN DESIGN AND AN ACCOUNT OF RECENT STUDIES IN ST. LOUIS, MISSOURI

Discussion*

By Messrs. Charles Terzaghi, George Paaswell, E. G. Haines, Kenneth Allen, N. T. Veatch, Jr., and S. D. Bleich.

Charles Terzaghi,† Assoc. M. Am. Soc. C. E.—The fundamental assumptions on which this design is based, are as follows:

(1) That there is no arching effect in the back-fill;

(2) That Mr. Feld's formulas‡ can be used for computing the pressure acting on the top arch of the sewer;

(3) That the soil reaction produced by the weight of the sewer and the back-fill is confined almost exclusively to the footings of the top arch, while the invert arch receives practically none; and,

(4) That the invert arch takes care of the horizontal thrust exerted by the top arch.

Assumption (1) can be considered correct for all the back-fill above the crown of the top arch. However, below the level of the crown there will inevitably be an important arch effect in the back-fill confined within the space between the sides of the cut and the sloping part of the arch, of the same type as that within the loose fill in the conical bottom part of a grain bin. This fact upsets the validity of Mr. Sharp's conclusion that the vertical component of the forces acting on any section of the top arch equals the weight of the column of soil above this section. For the central part of the arch, the vertical component will be much larger and for the sloping part very much smaller than that assumed by Mr. Sharp.

As to Assumption (2), Mr. Feld's earth pressure formulas were exclusively derived from tests made with sands having little or no cohesion. The back-fill of the St. Louis sewer trench consists of excavated material, much of which is plastic, according to the paper. Hence, Mr. Feld's formulas cannot be applied for solving the problem in question.

Assumption (3) represents the very basis of Mr. Sharp's design. The footings of the upper arch are supposed to take up the major part of the soil reaction and the invert arch practically none. These *a priori* premises

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* This discussion (of the paper by Charles E. Sharp, Jr., Esq., published in August, 1925, *Proceedings*, and presented at the meeting of September 2, 1925) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Mass. Inst. Tech., Dept. of Civ. Eng. Cambridge, Mass.

‡ *Buildings*, Monthly Issue of *Engineering and Contracting*, May 23, 1923, pp. 1163–1165.

have obviously been suggested by the well-known fact that the soil pressure acting on the bottom of an elastic foundation girder decreases somewhat from the points of load application toward the center of the space between. However, the shape of the pressure distribution curve depends both on the compressibility of the soil and on the flexibility of the girder. The more compressible the soil and the stiffer the girder the more the distribution of the soil reaction becomes uniform. If the bottom of the sewer were a plane slab with a thickness equal to that of the invert arch, Mr. Sharp's assumption would probably be fairly correct. However, the bottom of the sewer consists of an arch; and an arch confined between unyielding abutments is almost as stiff as a girder the height of which is equal to the rise of the arch, regardless of whether or not it is hinged. As a consequence, the soil reaction will be practically uniformly distributed over the bottom of the sewer, including the back of the invert arch. Due to its considerable width the invert arch will take up the major part of the soil reaction and, as a consequence, will exert a horizontal thrust. What happens to this thrust? Certainly, Mr. Sharp would not trust the plastic soil to take it up. Hence, it must be resisted by the upper arch, and since, according to Assumption (4), the invert arch takes up the thrust of the upper one, the combination is like an attempt to construct a net profit out of two financial losses.

As regards Assumption (4), it is not possible to take up a horizontal thrust by tie-rods, which are embedded in an arch and, consequently, curved. One could as well utilize an arch as a compression member.

The method which has been used for calculating the St. Louis sewer may be briefly described as follows: The sewer represents a reinforced concrete tube, the calculation of which has led the designer to the conclusion that the bending moments produced by soil pressures and by soil reactions are very important. In order to reduce the construction costs, he provided his tube with two little wings (the projecting parts of the footings of the top arch) braced against nothing but the plastic compressible soil; in addition, he has weakened the invert arch at two points by separating it from the footings of the top arch by means of metal strips, believing that these trifling construction details fundamentally modify the distribution of bending moments throughout the whole structure. As a result he has unconsciously reduced the coefficient of safety of the invert arch from 4 to 2, or to 1.5, while the coefficient of safety of the upper arch practically remained unchanged. In the speaker's opinion, it would have been less troublesome and very much safer, to reduce the costs of the original design (that is, of the reinforced concrete tube design) by reducing the factor of safety of the whole structure from 4 to 3.

Besides not serving their intended purpose, the projecting parts of the footings of the upper arch involve a distinct disadvantage. The back-filled space above the projecting parts of the footings is limited by two practically impermeable materials, the concrete and the plastic soil. Within a short time these spaces will be transformed into permanent water-pockets, because the water cannot drain away. It will gradually soak into the clay, thus causing the clay to swell. The effect of this cannot possibly be predicted by the speaker, because different clays behave very differently in contact with water,

and he is not familiar with the St. Louis fire-clay. Some clays swell but little; others disintegrate rapidly and dissolve. Such clays would gradually be squeezed into the voids of the back-filling, which, in turn, would cause the street surface to settle. Other clays expand so energetically on moistening, that they would crush any reinforced concrete tube.

According to some interesting information courteously furnished by Carl S. Scofield, Agriculturist, U. S. Department of Agriculture, certain clays of South Dakota swell to such an extent that they overcome the tensile strength of foundation masonry and lift the upper part of structures several inches above its original level. As a result one may see horizontal cracks 2 in. wide between the stationary and the uplifted part of the foundation walls.

In Constantinople, the speaker has observed a retaining wall 30 ft. high with a back-fill composed of a mixture of stones and clayey top-soil. In the course of seven years, the retaining wall bulged out in one place about 2 ft., while the remainder of the wall retained its original position. The local displacement of the wall was found to be due to a leakage in a sewer located within the back-fill. The leakage caused local swelling of the clay, which, in turn, forced the wall out of plumb. Hence, in case the clay is apt to swell energetically, the most important part of the design would be to provide for efficient and rapid drainage.

The paper does not contain any information concerning the physical nature of the plastic soil, although this nature seems to be the only factor of importance—a factor which may upset the validity of any calculation. In addition, the paper does not mention the drainage problem; but at the same time it deals at length with the temperature stresses. This fact seems to be characteristic of the conceptions the author had concerning the relative importance of the various factors affecting the safety of his structure.

The speaker wishes to emphasize that his criticism by no means intends to minimize the merit of Mr. Sharp's painstaking efforts in searching for an economic solution of his problem. As a matter of fact, the paper represents a perfect and very commendable example of how such problems are handled the world over, by experts of every country, and the speaker has simply used it as a pretext for expressing his opinion concerning this practice in general. The most vicious part of it lies in the fact that the designers persistently ignore the utter uncertainty of the information available concerning the intensity of the soil pressure and the distribution of the soil reactions. By so doing they deceive themselves and others. Without being conscious of this fact, they ignore the true degree of accuracy of their calculations—the true degree of safety of their structures—and they kill the initiative for further improvement in its very germ.

The proper procedure should be more as follows: Whenever the expense warrants working out an economic solution of similar construction problems, two different sets of fundamental assumptions should be made, representing the extreme limits of the range of error in estimating soil pressures and soil reactions, and the reinforcement required for taking care of either one of the two systems of forces should be combined in the final design. In addition, the system of forces which act on a structure of the type of the St. Louis

sewer five weeks after it is completed, may be very different from those acting five years later, after equilibrium has been established between the homogeneous native clay, the plastic back-fill, and the water. This fact, too, should be carefully considered. Then and then only would the designers fully realize the uncertainty of the fundamental assumptions of their calculations. They would stop spending their time studying temperature stresses and structural refinements and they would concentrate their efforts on an attempt to collect more empirical data for the purpose of gradually reducing the range of error from 100 or 200 to, say, 30 per cent.

However, no evil can be remedied unless its existence is discovered, and many engineers seem to be still far from realizing the utter unreliability of the traditional assumptions concerning soil pressures.

George Paaswell,* M. Am. Soc. C. E.—Although the author makes a sincere attempt to get away from the ordinary methods of design with their fictitious assumptions of pressure distribution, there is a more or less partial surrender to these discarded theories in the final analysis.

Unless the exactness of analysis of arch design be correlated with the exactness of analysis of pressure distributions, the design becomes unbalanced. Take the concrete arch. The equations for moment are derived on the assumption of an arch rib, whereas the structure is a continuous one. The problem is analogous to the use of beam equations in designing plates. It is true that there are no data giving reliable methods for the design of an arch plate, but nevertheless it must be recognized that the rib equations will give greatly exaggerated stresses. On the other hand, the author has used a retaining wall equation in the calculation of the side pressures. The speaker cannot emphasize too strongly that there is absolutely no connection between the usual retaining wall loading and soil pressure distributions. The complicated equation used, with the meaningless slope angle, ϕ, has no significance in the design of this structure, nor, indeed, has it much significance in a conscientious design of a retaining wall. It has been well indicated that the latter problem is an indeterminate one, depending on the movement of both the wall and the foundation, as well as on the soil characteristics.

The problem is essentially the distribution of pressures through a clay envelope about a more or less circular bore; it requires essentially the study of a clay model under load, and not a study of a retaining wall equation. Each type of clay gives, under test, certain distinguishing load deflection curves, presenting a far better insight into the action of the base of the structure, than the rather bland assumption that a "load of 2.5 tons per sq. ft. would cause no lateral flow". The deflection of the arch under its variable loading permits adjustment of the clay envelope that can only be interpreted rationally after exhaustive tests upon a specimen. The construction of the sewer, affording a natural soil drain, may affect the water content of the clay, again making a vital structural change. Time effects on clay and changes in water content must be studied in the model and the results of data obtained from such experiments must be incorporated into the accepted design.

* Asst. Engr., Board of Transportation, New York, N. Y.

To conclude, the speaker feels that it would have been well worth while to construct a model of the structure and the clay envelope and to conduct a series of tests, giving empiric relations far more important than the accepted equations of design.

E. G. HAINES,* M. AM. SOC. C. E.—The question of soil resistance enters largely into this problem and the fact needs emphasis that engineers are not justified in assuming that either the active pressure or passive resistance of a soil in its natural state and position may be determined by any rules resulting from tests made on soil placed as an embankment. The reason for this should be apparent. Natural soil is a more or less compact concrete substance, whereas an embankment is composed of not only granular but granulated discrete particles, and only after a lapse of time or the application of pressure does such soil begin to present the characteristics of natural soil. Even with sand, the statement holds true in nearly all cases. While it merits more extended treatment, the crux of the matter may be summed up in one brief sentence: The bearing resistance of a soil depends, not on its kind but on its condition. For example, for an identical clay, the resistance may vary from many tons per unit of area down to zero, depending on the water content alone. Although very important, water is only one of the modifying influences affecting soil. Grain size and colloidal content are also of extreme importance, as has been reported to the Society by the Special Committee on the Bearing Value of Soils for Foundations, etc.

After approximately ten years of active effort to collect from engineers records of tests and settlements on soils, the Committee was obliged to abandon, at least temporarily, the attempt to "codify present practice on the bearing value of soils", as was reported to the Society† with a statement of the reasons for that action. In its report the Committee stated:

"In regard to codification, therefore, the Committee feels that these investigations have been carried along sufficiently to warrant the statement that information does not exist of a character, in a form, and of a degree of dependability which will permit the Committee to prepare a code showing present practice in the bearing capacity of soils, as defined by local names and terms; and that, as a basis for preparing such a code at some future date, it is necessary to define the methods of classification—both for the soil as a whole and in its component parts—and both the apparatus used and the method of making tests must be standardized, either in the form suggested by the Committee, or modified as the Society sees fit."

The underlying reasons for that conclusion and recommendations for the steps necessary to correct that state of affairs have been included in reports of the Committee, but may be indicated and summarized briefly by again quoting from the report‡ of the Committee relating to the matter of settlement of soil under load:

"Aside from incorrect or indefinite classification of the soil, much of the present uncertainty, as to bearing capacity of different soils, is due to the fact

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* Asst. Div. Engr., Board of Transportation, City of New York, New York, N. Y.

† *Proceedings*, Am. Soc. C. E., October, 1923, Papers and Discussions, p. 1732.

‡ *Loc. cit.*, March, 1922, Papers and Discussions, p. 533.

that such tests as are recorded were made in a great variety of ways, often with crude devices and lacking sensitiveness, which precludes the possibility of measuring and recording the smaller compressions and elastic reactions under relatively light loads. Tests made in such a manner are practically valueless, for it should be understood that the bearing capacity of soil is determined, not by the settlement produced by a heavy load, for then displacement of the grains and failure of the soil have already taken place, and, when the settlement stops, the soil has a factor of safety of only 1 +, but rather by the elastic reaction of the soil and the measure of compression under relatively light loads, which is true of other materials used in engineering construction."

Soil is, and must be considered, an engineering construction material, and until it is subjected to the same processes of analysis as steel, timber, cement, and other structural materials, and its properties thus determined, it will remain impossible to lay down any rational basis for the design of structures supporting or supported by the soil. The present assumptions as to bearing values are based either on judgment only; on a limited number of tests on soils of unknown characteristics; or on the fact that certain structures on apparently similar soils have stood safely; but, in every case, the factor of safety is entirely unknown.

At almost every step in the work of the Committee it has been apparent that no uniform basis existed, either of tests, classification, or methods, on which a code could be prepared; and the Committee's plan of carrying along with the codification of bearing values, the investigation of soil physics, as authorized by the Board of Direction, and of laying down a basis on which future observations and tests may be made, in a form to give comparative results, has been well justified. The Committee was appointed, not only to codify present practice, but "to report upon the physical characteristics of soils in their relation to engineering structures". Following these latter instructions a number of important and interesting facts and discoveries have been presented to the Society in the reports of the Committee, and such work is being extended as far as assistance and funds are made available.

KENNETH ALLEN,* M. AM. SOC. C. E.—The economic advantage of reinforcement increases with the size of the sewer; in a large sewer the economy gained by the use of reinforcing steel is considerable, but with smaller dimensions there is some point where for practical reasons there is no economy in its use. In fact, there are some physical disadvantages connected with the use of steel; for instance, if the steel is accessible to water through the porosity of the concrete, it is liable to corrosion; and in design that matter is independent of considerations of first cost—it is a question of judgment rather than computation. The importance of placing the steel so that it is well protected by the concrete is unquestionably very great, for a bar misplaced so as to approach the inner surface and be acted on by the absorbed moisture will eventually corrode and rupture the adjacent concrete. Where the sewage is acid from industrial wastes or from its septic condition, or where it contains much salt, such action is relatively rapid.

* San. Engr., Office of Chf. Engr., Board of Estimate and Apportionment, New York, N. Y.

Another important question is in the quality of the concrete itself, both in the mix and in the use of a cement that is especially adapted by composition and hardness to resist the erosion of, or chemical disintegration by, the sewage.

N. T. VEATCH, JR.,* M. AM. SOC. C. E.—The question under discussion is a very important one to the Engineering Profession and the subject is worthy of careful study and investigation.

The use of the design as outlined by Mr. Sharp has unusual possibilities from the economic standpoint. A great many sewers based on this design have been and are being built in the City of St. Louis and a somewhat similar design is now being used in Kansas City, Mo., where, on 9 000 ft. of sewer ranging in size from 17 to 21 ft., the quantity of concrete was about 66% of that required for a monolithic arch and the steel about 80 per cent.

In using this design at Kansas City, it was decided to have at least 3 in. of concrete covering over all reinforcement. The design was used for all conditions whether in rock, earth or clay, or at places where pile supports were necessary, the shape of the footings being governed by the conditions encountered. In addition, rather conservative stresses were used, namely, 16 000 lb. per sq. in. for steel and 600 lb. per sq. in. compression in the concrete.

In the construction at Kansas City the materials are being analyzed with unusual care, as it is felt that sewers of this type probably require a little higher grade of concrete than those of heavier section. Also, in the design, a more massive structure has been used, although following the same principles as those at St. Louis. This was brought about by assuming conditions that might exist due to unequal back-filling and other such unusual circumstances.

S. D. BLEICH,† ESQ. (by letter).‡—The writer has had no occasion to design sewers of the immense sizes described by Mr. Sharp. The most striking thing in the design of these sewers is the slenderness of the arch construction as compared with sections used in sewer construction in Greater New York. This is probably the result of making the most of the lateral restraint or horizontal pressure of the surrounding earth and utilizing every advantage that can be obtained from steel reinforcement. The Sewer Bureaus in the various Boroughs of New York City use heavier sections for reinforced concrete arches, so as to minimize the possibility of cracks forming in the concrete due to tension. The invert of the sewer section given in the paper is much thinner than that of New York sewers. Moreover, the inverts of the latter sewers are to a large extent lined with selected or vitrified brick, as experience has shown that it is difficult to obtain good invert construction with concrete only; this lining may also give protection from the possible destructive action of sewage.

Some years ago while investigating dimensions for standard sewer sections, the writer had occasion to make a thorough investigation of a 16-ft. circular arch sewer by the elastic theory according to the method of Weyrauch. The crown thickness for this sewer was made 19 in. and the thickness of arch at

* Cons. Engr. (Black & Veatch), Kansas City, Mo.

† Asst. Div. Engr., Board of Transportation, New York, N. Y.

‡ Received by the Secretary, October 3, 1925.

the 60° joint was made 24 in. It might be of interest to compare the results of this investigation with those given by the author.

In arriving at the standard sections for circular sewers an exhaustive graphical and analytical investigation was made of a variety of sections for the different diameters. The loading considered was of two kinds: (1) A uniform load of 2 000 lb. per sq. ft. horizontally over the entire span; and (2) a uniform load of 600 lb. per sq. ft. for one-half the span and 2 000 lb. per sq. ft. for the other half. These loads were regarded as adequate for ordinary conditions. In locations where greater loads are to be expected, the sewer sections were sufficiently strengthened.

In Table 11 and on Fig. 20 the results of such an investigation are given for a 16-ft. circular sewer. It will be seen that for a uniform live load over the entire span, the line of pressure falls entirely within the middle-third and that for the one-sided live load, the line of pressure departs slightly from the middle-third at the radial joints 10°, 50°, and 60° to the vertical. The maximum compressive stress is low, being less than 150 lb. per sq. in.

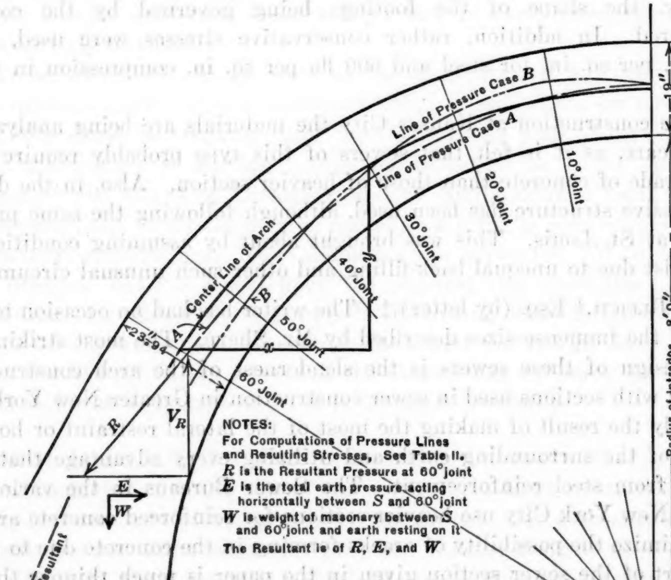

FIG. 20.—INVESTIGATION OF 16 FT. 0 IN. CIRCULAR SEWER, ELASTIC THEORY.

In designing the arch the portion between two radial joints making an angle of 60° to the vertical was considered. The portions below these radial joints were treated as parts of the abutments. In obtaining the trial thicknesses at the crown for plain masonry, the empirical formula:

$$d = \frac{1}{9}\sqrt{(s - r)\,10} + 0.02\,L$$

TABLE 11.—INVESTIGATION OF 16-FOOT CIRCULAR ARCH BY THE ELASTIC THEORY, AFTER WEYRAUCH.*

Radius of intrados = 8 ft.; crown thickness = 19 in.; radial thickness at 60° joint = 23.94 in.; rise of center line = 4.2929 ft.; span of center line = 15.582 ft.; radius of center line = 9.2162 ft.; average $\gamma = 0.27025$; $\epsilon = 0.007846$.

CASE A.—UNIFORM LOAD OVER ENTIRE SPAN OF 2 000 LB. PER SQ. FT.; H = 1 125 LB. PER IN. OF ARCH.

Radial nt, α.	CO-ORDINATES.		VERTICAL COMPONENT.	ELEMENTS FOR BENDING MOMENT.				ELEMENTS FOR NORMAL PRESSURE.			Radial eccentricity e, $\frac{M_a}{N_a}$, in inches.	STRESSES.	
	x, in feet.	y, in feet.	V_a, in pounds.	$V_{60°}\,x$, in foot-pounds.	$H\,y$, in foot-pounds.	$\frac{2\,000\,x^2}{2}$, in foot-pounds.	M_a, in foot-pounds.	$V_a \sin a$, in pounds.	$H \cos a$, in pounds.	N_a, in pounds.		Intrados, in pounds per square inch.	Extrados, in pounds per square inch.
60°	0	0	1 299	0	— 0	0	—1 819	1 124	563	1 687	—1.07	89	61
50°	0.731	1.001	1 177	11 390	—13 517	— 534	—4 480	901	723	1 624	—2.76	125	19
40°	1.867	2.137	987	29 092	—28 857	— 3 486	—5 070	635	862	1 497	—3.39	136	4
30°	3.183	3.058	768	49 598	—41 293	—10 132	—3 646	384	975	1 359	—2.68	120	14
20°	4.639	3.737	525	72 285	—50 463	—21 520	—1 517	179	1 057	1 236	—1.22	87	39
10°	6.191	4.153	267	96 468	—56 080	—38 328	+ 241	46	1 108	1 154	+0.21	56	64
0°	7.791	4.293	0	121 399	—57 971	—60 700	+ 909	0	1 125	1 125	+0.81	44	74

CASE B.—DEAD AND LIVE LOAD OF 2 000 LB. PER SQ. FT. OVER LEFT HALF OF SPAN AND DEAD LOAD OF 600 LB. PER SQ. FT. OVER RIGHT HALF. H = 782 LB. PER IN. OF ARCH.

Radial nt, α.	x, in feet.	y, in feet.	V_a, in pounds.	$V_{60°}\,x$, in foot-pounds.	$H\,y$, in foot-pounds.	$\frac{2\,000\,x^2}{2}$, in foot-pounds.	M_a, in foot-pounds.	$V_a \sin a$, in pounds.	$H \cos a$, in pounds.	N_a, in pounds.	Radial eccentricity e, in inches.	Intrados	Extrados
60°	0	0	1 128	0	0	0	—6 499	977	366	1 343	—4.83	124	—12
50°	0.781	1.001	1 006	9 896	— 8 786	— 534	—5 918	771	453	1 224	—4.83	125	—16
40°	1.867	2.137	817	25 273	—18 757	— 3 486	—3 464	525	560	1 085	—3.19	97	5
30°	3.183	3.058	598	43 088	—26 841	—10 132	— 379	299	633	932	—0.41	52	41
20°	4.639	3.737	355	62 798	—32 801	—21 520	+1 983	122	687	809	+2.45	10	73
10°	6.191	4.153	96	83 808	—36 452	—38 328	+2 534	17	720	737	+3.46	— 4	82
0°	7.791	4.293	— 171	105 469	—37 681	—60 700	+ 594	0	732	732	+0.81	29	49

* For method, notation, etc., see "The Elastic Arch," by Jakob J. Weyrauch, 1897, and Fig. 20.

was used, in which, d is the crown thickness, s the span, and r the rise, in feet, and L is the total load, in pounds per square foot. The thickness of the radial joints at the haunches was made equal to 1.26 d.

In this particular design an effort was made to have the line of pressure fall within the middle-third as far as practical without considering the effect of side pressure from the back-fill. Under the conditions encountered in New York, such as the existence of tall buildings with a number of floors below the street level, it does not appear safe to design structures so as to depend on the lateral support of the soil.

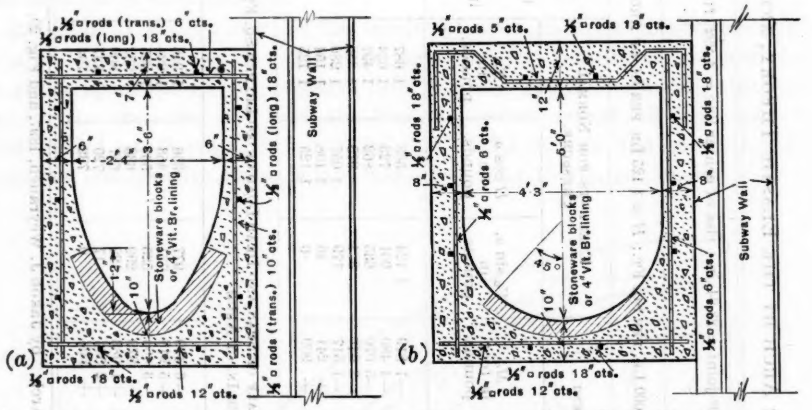

FIG. 21.—TYPICAL SECTIONS FOR SEWERS ALONG NEW YORK SUBWAYS.

Because of the great cost of excavation and of the limited space frequently available, it has been necessary to design sewer sections adjacent to subways of the required capacity with a minimum width. This has been accomplished by making the sections of reinforced concrete throughout with flat roofs, so as to avoid the thrust of an arch roof. Two illustrations of such sections are shown, in Fig. 21 (a) and (b), one with an egg-shaped invert and the other with a semi-circular invert.

POWER DEVELOPMENT IN THE MIDDLE WEST

Discussion*

By Harry K. Rubey, Assoc. M. Am. Soc. C. E.

Harry K. Rubey,† M. Am. Soc. C. E. (by letter).‡—The writer congratulates Mr. Abbott on his analysis of power development and wishes to elaborate somewhat on the future developments mentioned in the last paragraph of the paper.§

The increasing demands for electrical energy, including railway electrification, call for thought and plans to meet this growth with a minimum of wasted and duplicated effort in power-plant and transmission-line construction. Some advocates of "super-power" and "giant power" systems believe that a carefully preplanned system is the best procedure, while many of the present operators feel that gradual interconnection and development will best meet somewhat indefinite future needs. Where such a future need can be foreseen, it will doubtless pay to plan ahead to meet it.

If the railways contemplating more or less extensive electrification will co-operate with the power companies in pre-planning power-plant sites and transmission lines, considerable savings in capital expenditures will be realized and a more favorable load factor attained. Only a small percentage of the railways will electrify in the near future, but that small percentage will form a considerable part of the demand in certain territories.

The largest savings occur where it is feasible to construct commercial and railway transmission lines combined as one line along or adjacent to a railway right of way. Savings in capital expenditures and in operating expenses result from decreased right-of-way costs and from the ideal availability of transportation for construction and operation. Such a line will serve the industrial community at its end, the various communities along the railway, and the railway itself. Additional savings arise where these intermediate communities can be served from railway sub-stations.

Such combined systems will gather energy from both steam and water-power units. In water power developments there are periods of deficient run-off which will seldom coincide with maximum demands for railway, industrial, and municipal power. In many cases the seasonal demands from these various consumers will not occur simultaneously. Again, the hourly peak for

* This discussion (of the paper by W. L. Abbott, Esq., presented at the meeting of the Power Division, Chicago, Ill., July 11, 1923, and published in September, 1925, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chairman, Dept. of Civ. Eng., Univ. of Missouri, Columbia, Mo.

‡ Received by the Secretary, September 14, 1925.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1339.

railway operation can be scheduled to some extent on off peak-load hours. All this tends to produce a high load factor with a resultant low power rate.

The requirement of large capital expenditures restricts adoption of electrification by railways. It is possible that the power companies, with their better financial standing, can finance these combined railway and commercial transmission lines along or adjacent to railway rights of way and by thus minimizing the capital expenditures of the railways, can encourage their electrification.

A glance at the map (Fig. 2*) shows the very large number of small plants, most of which are wasteful of fuel and expensive to operate. It is these small plants that will be replaced first by large steam power stations at the mines and by hydro-electric power stations. Many of these small inefficient plants are found along the railways and would be served by the combined system as suggested. Economic pressure and governmental regulation (from the viewpoint of rate reduction and fuel conservation) will tend to do away with these smaller plants.

It is to be hoped that the railways which contemplate electrification will co-operate with the power companies before plans for generation and transmission are crystallized, to the end that minimized capital expenditures, lower power rates, and conservation of fuel may be attained.

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1337.

THE ST. LAWRENCE WATERWAY TO THE SEA

Discussion*

By Messrs. W. M. Black, G. B. Pillsbury, and L. F. Harza

W. M. Black,† M. Am. Soc. C. E. (by letter).‡—This paper and its discussions give many interesting and more or less well-founded figures of costs and of expected benefits. They also show the marked diversity of conclusions reached.

The entire subject is being re-investigated by an able board of American and Canadian engineers. Doubtless its report, when submitted, will contain data which will be as full and exact as are now obtainable. The writer believes that it would be wise to await the publication of that report before attempting to form definite judgments on the many controversial questions involved.

As stated by Mr. Sabin,§ the commerce of the Great Lakes causes for the people of the United States a saving of \$100 000 000 annually in transportation costs. If the contention that the proposed deeper St. Lawrence Waterway would result in similar transportation economies is well founded, the desirability of the improvement is established—provided the net cost of making the improvement is not too great.

In general terms it is proposed to regulate the lake flow by suitable works and to concentrate the general slope of the St. Lawrence River into a series of falls, by means of dams, causing an increase of navigable depth in the pools above the dams and utilizing the head formed at the dams for the production of power. In certain localities, where the pool depths are insufficient, side channels are proposed for navigation. It is manifest that, within certain natural limits, the higher the dams, the greater their cost as balanced against the deeper navigable channels of the pools and the greater possible power production.

It is admitted that there is an immediate market for additional power. The question of how quickly an increase of power to the full potential of the fall of the St. Lawrence could be sold is as yet unsettled, and is dependent in a measure on the cost. This, in turn, is a function of the depth of the navigable channel desired.

In so far as is known to the writer all the projects submitted have had for an object the provision of a navigable channel of a definite capacity, with a

* Discussion of the paper by Francis C. Shenehon, M. Am. Soc. C. E., continued from November, 1925, *Proceedings*.

† Major-General, U. S. A. (*Retired*); Cons. Engr. (Black, McKenney & Stewart), Washington, D. C.

‡ Received by the Secretary, October 19, 1925.

§ *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1708.

corresponding power development. Opinions differ as to the channel capacity required. The project depths considered have been 25 and 30 ft. It has been claimed that 25 ft. is an uneconomical depth for ocean carriers and *per contra* the claim is made that there would not be sufficient commerce to justify the cost of a 30-ft. improvement.

A study of this shipping question will disclose the following facts:

1.—For long voyages, when cargoes are available, the greater the cargo capacity of the ship, the less will be the cost per ton of transportation. It is due to this that the average tonnage of both ocean and lake vessels under construction has increased from year to year, and that the relative number of ships of small tonnage is growing less.

2.—For a given tonnage, and within proper limits, an increase of draft provides for a stronger and more seaworthy ship than an increase of beam. Structurally, the ship resembles a truss in this respect.

3.—The strains to which a ship is subject are much greater on the ocean than on the Great Lakes. Lake vessels can be built more lightly, and, consequently, more cheaply, than those designed for ocean navigation.

4.—The ship's complement of men as required by law is greater for the ocean than for the lake carriers.

5.—The draft of the ship is somewhat greater in fresh water than in salt water. For example, a 10 000 dead weight ton cargo steamer which draws, loaded, 27 ft. 2.75 in. in salt water, will have a draft of 27 ft. 6.2 in. in fresh water, or, in other words, with the same draft she can carry 161.5 long tons of cargo less in fresh than in salt water.

For the foregoing reasons transportation costs for a lake carrier in the lakes and in the Upper St. Lawrence River will always be less than for an ocean carrier in the same waters. *Per contra,* ocean carriers of larger tonnage, drawing from 26 ft. to 30 ft. are, to-day, the most economical means of transportation in the ocean highways.

It is claimed that a ship can be designed and built which will be equally good in the lakes and on the ocean. To the writer this seems doubtful. The conditions of navigation differ too widely. Certainly, a channel depth of more than 25 ft. would be necessary for a long-voyage ocean carrier. Such a depth would require expensive and extensive changes in the lake harbors and channels, as well as expensive works in the St. Lawrence River.

There exists to-day an established and efficient freight service between the interior of the United States and its Atlantic seaboard and, *via* the seaboard ports, with all parts of the world. Traders are conservative and a change of trade routes can be established only when the new routes offer positive and marked economies. Witness the slow growth of the Panama Canal traffic where the canal route provides enormous economies in time and costs! To make a deeper St. Lawrence Waterway a great trade highway, the saving resulting from its use must be manifest and marked.

In considering the project for a deeper St. Lawrence Waterway, the liabilities are (1) the first cost, and (2) the cost of operation and maintenance; the assets are (1) the power produced, and (2) the savings resulting from a lessened

transportation cost for traffic between the interior, the Atlantic seaboard, and the world at large. The costs to the people of the United States and Canada must be weighed against the benefits to be received. The cost items can be estimated closely; the items of benefit do not admit of exact calculation.

Under these conditions would it not be well to advance slowly? Cannot a project be formed which will permit a progressive development, limiting the works to be constructed at first to those necessary to constitute a fair test of the economic advisability of the entire plan?

The experiment of connecting the lake cities with the Port of Montreal by a line of boats through the St. Lawrence has been tried. The limited channel capacity, however, requires boats that are too small to provide the economic transportation necessary to enable the fixing of the combined lake, river, and ocean freight rates, including transfer charges at Montreal, at a figure enough below the existing rates (by rail to the seaboard and transfer there to the ocean carrier) to be attractive. The experiment was a failure.

Carefully made estimates show, however, that by using the large type of lake carrier to Montreal and transferring there to an ocean carrier the costs of transportation to Atlantic Coast cities or to Europe, plus those of the transfer at Montreal, are considerably less than by the present routes.

The building up of new trade routes requires time. It is improbable to-day that any one of the lake cities is prepared to supply or to absorb the cargo of an economical European carrier in package freight, with the short interval between successive trips necessary in such a trade. The cargo volume is too great. Later, should such a trade route prove to be advantageous, the traffic might grow to truly great proportions. The combined traffic of several of the lake cities might reach a sufficient volume at once. The 4 000 to 6 000-ton lake carriers could serve one or more cities economically and the combined cargoes could supply the ocean carriers. In earlier days it was quite customary for outgoing cargoes to be collected in the warehouses of a port throughout a week or more, until enough tonnage was assembled to load a ship for the port of destination. This is no longer the custom. Ships sail from the great seaports at frequent intervals for all parts of the world. An interior manufacturer to-day ships his products just in time to reach the port before sailing day and avoids storage delays and charges. This fact is one of the causes of the decay of small ports and of the growth of great ports. The port warehouses of to-day are used mainly for imports awaiting sale.

With the lakes and their hinterland to provide and absorb cargoes, there is no reason why the well-equipped and well-managed Port of Montreal should not become an ideal transfer point.

It will be noted that the writer is not considering the transportation of bulk freight. Much grain moves now by water to Montreal, but the volume available for export from the United States grows less as the population increases. Part cargoes of grain would doubtless be carried from time to time to make up for a deficiency of package freight. The traffic that would justify the deeper St. Lawrence Waterway must be of a character more similar to that of the Atlantic ports than to the bulk of the present lake traffic.

The cost of fitting the St. Lawrence River for lake boat navigation would not be excessive. It would be but little in excess of the value of the power produced. With such an improvement a thorough trial could be given to the project of direct water carriage between the lake ports and Europe and a new trade line built up. Later, if the increased expenditure for increased channel depths were shown to be justifiable, the deeper channel project could be carried out.

G. B. PILLSBURY,* M. AM. SOC. C. E. (by letter).†—This discussion relates to those parts of the author's paper dealing with the regulation of the Great Lakes; its purpose is to point out the engineering problems that must be solved before the economic feasibility of such regulation can be established, and before the plans for the necessary works can be drawn up, should regulation be found feasible.

Many members of the Society have regarded it as almost axiomatic that the regulation of the Great Lakes is inherently economically feasible; therefore, the writer may seem either bold or reactionary in asserting that the feasibility is yet to be proved. A consideration of the limitations that must be imposed on the levels of the lakes will, the writer believes, show that their regulation is not as simple and obvious a question as is sometimes assumed.

The interests concerned in the regulation of the Great Lakes are lake navigation, navigation of the St. Lawrence, power, and the owners of lands and improvements along the shores.

Obviously, lake navigation is interested in an increase in lake levels, that it may carry greater drafts through the channels connecting the lakes and into the harbors. Its greatest concern is the elimination of the extreme low levels, such as are now occurring. Within limits, present lake navigation is benefited by an increase in level at any stage of the lakes, as the short voyages and thorough organization of the traffic permit the utilization of the full draft available at the time. Lake navigation, however, would be adversely affected by any such increase in level as would hamper the operation of the loading or unloading wharves; for delays in loading and unloading are exactly as costly as delays and reduced cargoes en route. Navigation interests look also with extreme suspicion on any works in the connecting channels that would introduce delays to shipping, or that would tend to increase the hazards of navigation.

Navigation on the St. Lawrence is concerned with the minimum discharge from the Great Lakes System. The great Port of Montreal is not a tidal port, but a river port. There is about 20 ft. of fall in the St. Lawrence below Montreal, and the water level in Montreal Harbor depends on the volume of the discharge from the St. Lawrence and the Ottawa Rivers. The channel depths to and in Montreal Harbor have been won by the expenditure of millions of dollars in dredging, and are not lightly valued.

Power is obviously concerned, eventually, with increased minimum discharges. As long, however, as existing power installations use only a fraction of the minimum discharge of the river there can be no object in increasing

* Lt.-Col., Corps of Engrs., U. S. A.; U. S. Lake Survey, Detroit, Mich.

† Received by the Secretary, October 23, 1925.

this minimum discharge; nor will an increase benefit power until the development at some point on the river has reached a capacity corresponding to the minimum existing discharge. Considered from the economic viewpoint of the world at large, it will not be economical to expend large sums for regulation to increase the minimum flow for power purposes until all the developments on the river are beginning to reach the limit possible with the present minimum discharges.

The last interest concerned in the regulation of the lakes is the riparian owner, and this interest has sometimes been overlooked. At first glance it might seem that the increased lake levels proposed in any scheme of regulation that has been advanced are so small in amount, reaching a few feet only, that they could have no effect on the lands and the populous cities that border the lake. The investigation thus far conducted indicates, however, that the contrary is the case. Where land is valuable, improvements have been pushed beyond the absolute safe limit, as indicated by prior high water, to the reasonably safe limit, which would be surpassed by high water only at long intervals. There are many localities where a recurrence of past high waters would back up water in sewers, inundate the machinery pits of unloading wharves, and flood improved property in general. There is also the question of increased erosion of shore property at high lake levels.

The improvements effected have been made, in many cases, with full knowledge that they are subject to occasional damage by high water. It may be expected that the owners will insist that the frequency of such high waters will not be increased.

When the regulation of Lake Superior was under consideration by the International Joint Board, hearings were conducted to determine the fluctuations of the lake permissible under regulation. The so-called "high water of 1838" on Lake Superior is recorded at Elevation 605.32. From 1860 to 1914 the mean monthly level of the lake had been above 603.6 on four occasions, reaching in one month above 604. Nevertheless, riparian owners concerned, including municipalities and corporations owning wharf facilities, established at the hearings that any level above 603.6 would seriously injure their improvements, and the scheme of regulation prescribed by the Joint Board provides that the maximum regulated level of Lake Superior shall not exceed 603.6, except under such conditions of supply as would normally produce higher levels, in which case the levels should not exceed the level that occurred in the unregulated lake.

It may be confidently asserted that the riparian owners on any lake will impose a similar restriction on the regulated levels of the lake; that is, these will be limited to some level for normal operation which is lower than the actual highest levels of the past, with the provision that above this limit the lake must never rise oftener or higher than it would rise, with the same supply, in Nature.

The writer has seen the view advanced that the riparian owners would have no legal claim for damages provided that the maximum levels heretofore reached were not surpassed as a result of regulation. He is not sufficiently versed in law to express an opinion whether damages would lie if the fre-

quency of high waters were increased; but he is convinced that the governments of the two countries concerned would never sanction wilful injury to a considerable number of their respective citizens.

As a consequence it is necessary to increase the low levels of the lakes for the benefit of navigation, without increasing the high levels, and to manipulate the reduced reservoir capacity remaining so as to improve the minimum outflow.

A brief consideration will show that this can be made possible only by increasing the discharge capacity of the outlets of the lakes to be regulated. With increased discharge capacity it will be possible to hold the lakes at higher minimum and mean levels, and, at the same time, to discharge an excess supply, whenever it may arrive, without raising the lake above the level to which it would have risen, in the state of Nature, with such supply. It is because the power canals at the St. Marys River provided an ample excess discharge capacity that regulation of Lake Superior, made necessary by the power works themselves, became feasible. A large increase in the discharge capacity of the St. Lawrence at the control section at the Galops is an inherent part of any future improvement of the International Section of the river for power and navigation, and, consequently, the regulation of Lake Ontario will naturally be combined with such an improvement.

It may be well to correct any misconception as to the operation of the regulating works at the outlet of Lake Superior. These works were, in their conception, for the purpose of diverting the water from the St. Marys River into the power canals. Since no such diversion works can automatically restore the natural regimen of discharge, they are of necessity operated under governmental control. The operation is so directed as to decrease, in ordinary years, the fluctuations in the level of Lake Superior, while affording power users practically the entire river flow during dry years. These beneficial results are obtained, however, at the expense of increased fluctuations in the discharge from Lake Superior, made permissible by the existence of the enormous equalizing reservoir afforded by Lakes Huron and Michigan. The scheme of regulation, in the preparation of which the late Alfred Noble, Past-President, Am. Soc. C. E., had a prominent part, does not contemplate any hold-over storage from year to year. On the contrary, it provides for drawing down the lake each year in advance of the wet months in such manner that the inflow during those months can be safely handled.

The economic feasibility of the regulation of the lakes in general hinges, therefore, on the following questions:

(1) To what levels may the respective lakes be normally held without substantial damage to riparian interests?

(2) With an economically feasible outlet enlargement, what degree of improvement of lake levels and of discharge can be realized by regulation?

(3) Will the improvements realized be worth the cost of the outlet enlargement and of the works necessary to control the flow?

(4) What type of works, if any, acceptable to navigation interests, can be placed in the St. Clair River to control the outflow from Lake Huron?

The first question can be answered only by a thorough canvass of the situation at the principal harbors and towns along the several lakes.

Question (2) involves a computation of the cost of varying amounts of channel enlargement, with consequent increases of flow. It does not require much investigation to show that for an outlet such as the St. Clair River the cost mounts quickly with the increased discharge and soon becomes clearly prohibitive. The discharge through the St. Clair River during the winter months will certainly be less than the summer discharge, whatever the enlargement proposed; and it appears certain that the discharge capacity of the St. Lawrence during the winter months will also be limited, whatever the form and extent of the improvement. In any event no scheme of regulation can be considered that would increase the flow down the St. Lawrence at the time of the spring break-up, when, at Montreal and below, the river is packed with gorged and shoving ice.

With the limitations established, it then becomes possible to formulate rules for regulation, to test them by applying the supply factors of the several lakes established by past records, and to ascertain what improvement in present condition, both as to levels and as to discharge, can be realized.

In preparing and testing a suggested method for regulation the temptation is strong to assume that it will be possible to predict, in advance, the probable supply. Any scheme of regulation is necessarily based on prediction, which limits itself, however, to the assumption that the future will be no worse than the past, so far as maximum and minimum supply are concerned. To predict the future supply to a lake from the present rainfall records is a different, and the writer believes, an impossible task. The net water supply to the lake is the inflow into the lake, plus rainfall on the lake, less evaporation from the lake surface. Whatever hopes may be had that the inflow into a lake from its drainage basin may be predicted in advance from the rainfall records on its water-shed, no advance prediction can be hoped for regarding the variations in the rainfall on the lake and the evaporation from its surface. The latter two add to or diminish the supply as soon as they occur. Thus, the rains of August will have some effect on the inflow during September, but the temperature and humidity records of August will have no effect on evaporation during September. If it is realized that the average loss to Lakes Huron and Michigan from evaporation exceeds the inflow of the St. Marys River, it will be easy to see how uncertain any prediction of supply must be.

In estimating the benefits to navigation resulting from a proposed regulation (Question (3)), the cost of regulating works must be compared with the cost of securing a like measure of improvement by dredging. The author has shown the very large benefits that can be secured to navigation by deeper channels. It is quite immaterial, however, so far as navigation is concerned, whether depth is secured by dredging or by raising the stage. On the other hand, a free, open and unobstructed channel through the St. Clair River is an unquestionable asset to navigation, the value of which must be debited against any regulation scheme in proportion to the obstruction or constriction imposed by the works in the St. Clair proposed in that scheme.

In summary, the writer hopes he has shown that the economic feasibility of regulating the Great Lakes is not axiomatic, but that it can be established only after due and full investigation. The instructions to the present Joint Board of Engineers on the St. Lawrence Waterway require such an investigation, which is in course, and which should form a basis for a determination of this vital question.

About fifteen years ago a board of eminent engineers answered the question in the negative. Whether or not present conditions and new facts warrant a reversal of this prior finding is a matter that can well be given the best engineering thought of the Society.

L. F. HARZA,* M. AM. SOC. C. E.—It has been said by Guy E. Tripp, Chairman of the Board of Directors of the Westinghouse Company, that the general prosperity of any country or section of any country is directly proportional to the amount of mechanical power in use by that country. It is mechanical power only that makes the difference between the present and the time when a man worked 12 or 15 hours a day to make a living for his family, and when his wife spun the yarn and made the clothes, and the children worked from early childhood; it is only mechanical power that makes the difference between the standard of living and the average wage then and now. If that is the case—and it has been very well demonstrated—and if the eastern section of the United States is to receive all the benefit of this power development, why not let the Middle West have the benefit of the transportation development if there is any benefit to be derived.

* Cons. Hydro-Elec. and Hydr. Engr., Chicago, Ill.

MULTIPLE-ARCH DAM AT GEM LAKE ON RUSH CREEK, CALIFORNIA

Discussion*

BY MESSRS. L. R. JORGENSEN, WALTER H. WHEELER, C. A. P. TURNER, THOMAS H. WIGGIN, C. H. HOWELL, E. W. LANE, E. J. WAUGH, AND ALFRED D. FLINN

L. R. JORGENSEN,† M. AM. SOC. C. E. (by letter).‡—The disintegration of the middle zone of the arches in the Gem Lake Dam, has naturally given great concern to the writer, as he was responsible for the design and construction of this dam nine years ago; he has therefore been seeking earnestly for a true explanation of this rapid deterioration.

As concrete consists simply of rock, sand, cement, and water, the problem is limited to the investigation of these four materials, aside from the mixing and general workmanship. At the time of the construction of the dam and also as late as June, 1923, these materials were analyzed and tested frequently. The following notes on the tests are taken from a report made by the Noble Inspection Bureau (now Abbot A. Hanks, Incorporated, San Francisco, Calif.):

"Water from Gem Lake	Parts per Million
Sulphates (as Na_2SO_4)	None
Chlorides (as NaCl)	2.90
Carbon dioxide (CO_2)	2.20

"As the sulphates, chlorides, and carbon dioxide were present in such a very small amount, it was not necessary to make further analysis in connection with the investigation of deposit on the concrete. This water is unusually pure."

Relative to crushed rock, the report states:

"We [Noble Inspection Bureau] treated the sample of crushed rock to determine its solubility in water. The results indicate that the calcium carbonate deposit on the down-stream surface of the dam does not originate from this rock."

The sand used was found along the shore of the natural Gem Lake, which was lowered about 15 ft. when excavating for the dam foundation. Approximately, one-half this sand required washing in order to eliminate excesses of clay and vegetable matter. Both mortar briquettes and compression specimens were made of the local sand and of standard Ottawa sand, and the local sand was washed whenever it gave results below the standard in compression and below 90% of the standard in tension. Three kinds of sand were used: A

* Discussion of the paper by Fred O. Dolson and Walter L. Huber, Members, Am. Soc. C. E., continued from September, 1925, *Proceedings*.

† Cons. Engr., Constant Angle Arch Dam Co., San Francisco, Calif.

‡ Received by the Secretary, September 14, 1925.

semi-silicious, a basaltic, and a granite sand from the rock crusher. The latter was mixed with the sand from the two natural deposits.

The original structure was built by the Duncanson Harrelson Company, of San Francisco, which company had had large experience in concrete work and built a good structure. The water contents were kept to a minimum for reinforced water-tight concrete. It is true that the crushing strength of the concrete was not quite as high as ordinarily would be expected for the richness of the mix used. There are two reasons for this: First, concrete in reinforced arches which has to be water-tight can not be deposited as dry as required for maximum strength. In a gravity dam where men can work directly on the newly deposited concrete a slump of 3 in. can be used, but for reinforced concrete in forms, where the concrete can only be reached with long-handled tools, a slump of approximately 5 in. must be used to obtain water-tight concrete. Secondly, the shape of the crushed rock was somewhat elongated. The stresses used in the design were conservative, and therefore the structure was never over-stressed due to the water load.

When the dam was first subjected to water pressure it was remarkably tight. The first time the writer observed anything wrong was about three years after completion. A very heavy white deposit consisting of practically pure calcium carbonate was noticed on the down-stream face 30 ft. below the crest. It is almost impossible entirely to avoid this white deposit on the down-stream face of the dam, at least in moist spots, but in this case the deposit was too heavy to be harmless and kept increasing.

The small white deposits often seen on the down-stream face of dams originating merely from the leaching of the free lime in the cement, or from whatever laitance there may be in the construction joints, are harmless, but when these materials have leached out and appear on the down-stream side in the form of calcium carbonate, the process should stop and nothing further should happen. In the case of the Gem Lake Dam the process never stopped; it seems that in the zone subject to water pressure the cement itself broke down during the coldest weather. Where else could the large deposit come from?

The cement had passed the usual test. It might, however, have been slightly underburned without being noticed, causing it to be at the point of unstability when exposed to the severe cold. Under ordinary conditions the concrete undoubtedly would have kept its strength, but under the severe climatic conditions imposed on it in the middle zone, it deteriorated and lost its entire strength.

The deterioration started at the construction joints, as they were naturally the weakest points in the structure, but if the leaching had only been confined to whatever laitance or free lime there might have been in the joints, the process would soon have stopped. Fig. 7* of the paper shows clearly that the deterioration of the concrete was not confined to the joints. A zone 30 ft. high, beginning 30 ft. below the crest and extending all the way across the structure, was subject to heavy frost while the water level fluctuated between

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1321.

the two levels. In this zone the concrete is practically uniformly deteriorated, whereas above and below it is still in good condition.

If the sand had been the cause of the poor showing of the concrete, it could be expected that there would have been good and bad spots of the material, as three kinds of sand were used, sometimes washed and sometimes unwashed.

An analysis of the deposit on the Gem Lake Dam shows the following:

	Percentage
Silica (SiO_2)	0.76
Iron (Fe)	0.05
Alumina (Al_2O_3)	0.39
Calcium carbonate ($CaCO_3$)	97.44
Magnesium carbonate ($MgCO_3$)	0.11
Total	98.75

showing that the deposit is practically all calcium carbonate.

Inasmuch as the deposit continued to accumulate, it was, in the writer's opinion, due to the cement breaking down gradually and the lime in the cement being leached out continually. The time of actual breaking down was during the coldest weather.

The Agnew Lake Dam, built of practically the same material on the same stream, but 500 ft. lower in elevation, is still in first-class condition. The water level in this reservoir, however, is not appreciably lowered during the cold season, and snowdrifts protect the down-stream face to a great extent from the extreme cold weather.

Three years ago (1922) the writer acted as Consulting Engineer on a multiple-arch dam then being built in Northern Sweden on the River Luleaa, 50 miles north of the Arctic Circle. Two dams of a design similar to the Gem Lake Dam were constructed at a location where at times the temperature drops to 50° Fahr. below zero. Both dams have been in use for two years; they are being carefully watched for frost action, but so far none has appeared.

These dams are provided with a back-fill of earth on the down-stream side for a short distance up. The back-fill was put in to protect the lower arch portion against the very low temperature and consequent contraction near the footing where the arches are fixed to the rock. The upper portion of the down-stream face is still uncovered and, therefore, open to inspection. The water level fluctuates during the cold season.

On recent designs of multiple-arch dams, the writer has favorably considered arches of 50 ft. span, instead of 40 ft. A greater thickness of arch is necessary, but this should make it possible to keep the construction joints tight, just by careful workmanship, without plastering the up-stream side. When an arch is so thin that heavy frost can penetrate the concrete and cause freezing on the water side below the water level in the reservoir, it can hardly be expected that a plaster face will have a long life; the adhesion is seldom 100%, and reliance for water-tightness therefore should be placed as much as possible in the concrete itself.

WALTER H. WHEELER,* M. AM SOC. C. E. (by letter).†—The description of the Gem Lake Dam is interesting, but the disintegration of the concrete in the structure by frost can be readily explained.

In the description of the aggregates two facts are mentioned which would account for the difficulty. One of these facts is that the lake sand used in the concrete contained 3½% of clay and 1% of dirt, and the other that the screenings from the rock crusher were mixed with the lake sand in the proportion of 1:3. Presumably, the blue limestone, referred to as the country rock at the dam site, was used in the concrete without removing the dust which resulted from the crushing operation. It is the writer's experience that concrete made of aggregates which contain clay and dirt will not resist frost action, and also that limestone dust is very detrimental to the durability of concrete, also that concrete made with certain varieties of limestone aggregate will be quickly disintegrated by frost. The writer has observed many examples of the disintegration (by frost) of concrete in which similar aggregates have been used, and also many examples of concrete made with clean durable aggregates, which have resisted the frost under equally trying conditions. He does not hesitate, therefore, to assign at least a part of the trouble on the Gem Lake Dam to the aggregates used in the concrete. If concrete is to be exposed to frost the aggregates must be carefully selected with that possibility in mind.

There may also be other contributory causes which are not so apparent from the published description.

C. A. P. TURNER,‡ M. AM. SOC. C. E. (by letter).§—This paper should prove of much value to the profession in calling pointed attention to that particular kind of concrete which will not withstand frost action, namely, concrete compounded of sand as fine aggregate containing 3½% of clay and 1% of dirt. The broad conclusion that concrete disintegrates in like manner when made with suitable aggregate is not warranted from this example. Good initial resistance to compression of sample specimens is too often mistaken as a determinant of frost resistance although in no wise related thereto.

For purposes of discussion, the concrete utilized in the construction of dams may be classified as (*a*) structural concrete; and (*b*) mass concrete. Hollow dams (such as the original Gem Lake design), flat slab dams, and the like represent Class (*a*), while the mass concrete of the gravity type present examples of Class (*b*).

That frost disintegration occurs in Class (*a*) concrete because of the relative thinness of the section, but does not occur in Class (*b*) concrete because of its greater thickness, is apparently the deductions of the authors from the single experience described and discussed.

In certain slabs of reinforced concrete dams much thinner than the Gem Lake section and exposed at times to temperatures of 50° below zero, or lower, no material disintegration by frost has occurred in twelve years or more, whereas in some structures of the gravity type with far thicker sections and

* Designing and Cons. Engr., Minneapolis, Minn.

† Received by the Secretary, September 8, 1925.

‡ Cons. Engr., Minneapolis, Minn.

§ Received by the Secretary, September 12, 1925.

in which care had not been used to secure clean aggregate, the writer has observed disintegration by frost.

The importance of the question at issue lies in the fact that the structural concrete dam may commonly be safely built at a much lower cost than a dam of the mass concrete type. Many irrigation projects are rendered commercially feasible by using the economic dam structure while they would be lacking in feasibility with the more expensive gravity types. The disintegration of mass concrete, or the thinner section of concrete, by frost appears to the writer as one of progressive action which may render even the massive structure unsafe within a period shorter than its expected life. Hence, the need of care in the selection of concrete materials whether in the bridge pier or dam structure exposed to the elements. The additional first cost of washing sand and securing sound aggregate is always small compared to the cost of repairs when proper care has not been exercised.

THOMAS H. WIGGIN,* M. AM. SOC. C. E. (by letter).†—There is at present under construction from the writer's designs a multiple-arch dam about 300 ft. long and 33 ft. maximum height, and the subject is, therefore, of great interest to him.

Many concrete structures, including dams, are deteriorating with the aid of moisture and frost and it is generally impossible to fix the cause with certainty. The authors call attention for the first time, in a pointed way to the fact that the immersed side of a thin exposed wall of concrete is in the same danger from the formation of ice crystals in its pores under the influence of cold coming through from the dry side of the structure as the side not immersed of a thick, leaky wall.

Deterioration was expected and occurred in the mortar lining of uncovered steel pipe, but here the exposure was severe as only a thin plate of steel intervened between the outside cold and the saturated mortar lining only 2 in. thick.

Filter roofs, 6 in. or more in thickness, covered with about 2 ft. of earth, are often wet with condensed moisture and sometimes permit thin ice to be formed on the water below, showing that the air which touches them is, at least, as cold as the unfrozen water against a thin dam. The writer has never noticed deterioration in the lower surface of such filters. None of these filters was in a region of extreme cold.

In the United States some concrete stand-pipes have deteriorated outside from leakage and freezing, but the writer does not recall deterioration inside although the concrete was porous and had to be water-proofed on the inside to prevent further damage outside. The French have constructed some of these stand-pipes that seem to have no imperfections from leakage or frost. These stand-pipes, again, are not in semi-Arctic regions; also, there is more or less circulation of water.

The Stony River Dam in West Virginia is a hollow dam of the Ambursen type. It is at an elevation of about 3 000 ft. and experiences very severe winters compared with most places in the East. Ice forms on the inner faces of

* Cons. Engr., New York, N. Y.

† Received by the Secretary, October 7, 1925.

the hollow. Deterioration has occurred on the water face, although mostly high up in the zone of fluctuating water levels.

The writer has been advising on the repair of a gravity concrete dam. The concrete has deteriorated both on the water face and on the down-stream face. Disintegration down stream is apparently due to leakage and freezing; that up stream, similarly, is apparently due to saturation and freezing. The concrete is about 1:3:5 and rather poor. Several hundred thousand dollars will be expended in repairs. The safety of the dam has not yet been endangered by deterioration. The interior is sound. This is only one of many dams that have shown signs of surface disintegration. Only good stone masonry seems exempt, and that needs repointing in some cases.

It is not agreeable to contemplate that in spite of all care, concrete structures so often fail to endure. The writer started thirty years ago with the feeling that the place for concrete was underground. Unfortunately, stone for stone masonry is not often available at a reasonable price and the advent of concrete was the beginning of the decline in the supply of stone masons. Engineers are often forced to use concrete where its expectation of life may be comparatively limited, or at least doubtful, because stone would cost so much more that the concrete structure could be often renewed at a less total capital cost. "Hope springs eternal" also that vigilance will produce concrete of such quality that it will be among the structures exempt from major deterioration.

Among those who are engaged in research on cement and concrete are some who lay the deterioration of concrete to the use of too little cement. In the Gem Lake Dam only 1.5 bbl. per cu. yd. were used. Perhaps 1.75 or 2 bbl. would have shown different results. Some old structures with even less cement are still without blemish, it is true, but this may have been due to extraordinarily good sand and proportioning.

The writer chose a multiple-arch dam for the case mentioned at the beginning of his discussion because rock foundation was found near the surface and other conditions made it by considerable the cheapest type, except possibly a slab dam. About 1.6 bbl. of cement per cu. yd. are being used in the piers and 1.75 bbl. in the arch which is only 1 ft. thick. Even with this thickness the stress is only about 150 lb. per sq. in. from water pressure alone and it is still very low, with scarcely any tension, with extreme conditions of temperature. It is a flexible structure and no joints need slip to provide for temperature changes. It was hoped to make the dam tight by the excellence of the concrete, but in addition a complete water-proofing of asphalted fabric and asphalt, covered with mastic for protection against ice, is to be provided. The circulation of water in the reservoir will be fairly vigorous and the climate is that of the New York region, that is, generally free from prolonged intense cold.

In their struggles to avoid one set of difficulties, human beings are prone to run into others just as serious. Nature will not be altogether circumvented. An earth dam with generous slopes, paved water slope, grassed down-stream slope, masonry core, and well-drained toe comes as near to being a part of the landscape in permanence as can be produced by man, yet it requires a generous masonry spillway for its protection and perhaps fails by overtopping when the

record storm for the region occurs. The masonry dam even partly disintegrated may safely withstand overtopping at such a time.

C. H. Howell,* M. Am. Soc. C. E. (by letter).†—This paper is an exceedingly instructive one. The writer believes with the authors that "all the story is not yet told" concerning multiple-arch dams; he would also include all flat slab dams.

The smaller volume of concrete contained in these hollow types, as compared with ordinary gravity and arch sections, has often led to their erection at sites utterly unsuited to them. To expose thin concrete slabs or thin arches to high water pressures and low temperatures is inviting ultimate failure. It is a misuse of types of dams which have a definite, although rather limited, use.

Like all innovations, these hollow types of dams have their over-enthusiastic supporters who can see no merit in the older types and strenuously insist on multiple-arch or flat slab designs for every site. The history of the Gem Lake Dam will probably diminish some of this enthusiasm. The writer knows of one other dam, a flat slab structure, which is deteriorating in the same manner as the Gem Lake Dam. Both these dams were erected in climates that are too cold.

E. W. Lane,‡ M. Am. Soc. C. E. (by letter).§—Since many dams of the multiple-arch and Ambursen types have not been seriously affected by frost action, the trouble at the Gem Lake Dam does not seem to be inherent in dams with thin concrete walls. It does serve, however, to emphasize a possible source of weakness, and the necessity of much more than ordinary precautions in the construction of such dams, especially in cold climates.

A few years ago, the writer inspected a large gravity dam in which the concrete was disintegrating in a manner very similar to that at Gem Lake. The dam was constructed by competent engineers, but within a few years the concrete began to disintegrate and, when inspected, had scaled off in places to a depth of as much as 6 in. and could readily be dug with a knife for some distance from the surface, being about as resistant as compact gravel. The writer was informed that the cause of the trouble was the limestone aggregate. This had been brought from a considerable distance, as the local stone was unsuitable. It had passed all the ordinary tests, but an examination to determine the cause of the disintegration had revealed that the limestone was filled with microscopic veins of shale which broke down on exposure to the weather. Had this material been used in a dam with thin sections, it would certainly have necessitated reconstruction.

The fact that the buttresses of the Gem Lake Dam were not affected strongly points to the water as being an element in the cause of disintegration, but this does not prove that it was not the quality of the aggregate that was the primary cause of the difficulty. In his description of the construction of this dam, Mr. Jorgensen‖ states that, in the lower part, limestone from

* Engr., U. S. Bureau of Reclamation, Denver, Colo.

† Received by the Secretary, October 9, 1925.

‡ Detroit, Mich.

§ Received by the Secretary, October 13, 1925.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 868.

the outlet tunnel was used for the aggregate. Is it not possible that the limestone aggregate was the cause of the trouble, and that the reason the upper part was not affected was because it was constructed with granite aggregate?

The writer believes that the materials entering into the construction of a multiple-arch or buttressed dam should be subjected to a much more critical examination than for a dam of gravity section. He would suggest subjecting the aggregate and moulded specimens of concrete to a considerable number of alternate freezings and thawings. This should not be difficult with the small mechanical refrigerators which are coming rapidly into use. In northern climates it could be readily done in winter by natural means.

The danger resulting from disintegration of multiple-arch dams was called to the writer's attention by an inspection which he made of a dam of this type in Central New York. This dam (Fig. 12) consisted of three vertical arches supported by buttresses, flanked by an ogee gravity spillway section at one end and a gravity section abutment at the other. The concrete had disintegrated badly in places (Fig. 13), especially at the springing line of the arches and at a crack (probably caused by temperature) which extended down the crown of each of the three arches. The construction joints were also seats of attack. The disintegration was not confined to the arches, but included also the buttresses and the ogee spillway section. The writer understands that this dam has been repaired, but is not aware what method was used.

E. J. WAUGH,* M. AM. SOC. C. E. (by letter).†—In any discussion involving disintegration in concrete the first question that naturally arises is what was the quality of the concrete before deterioration commenced. If the up-stream face had been absolutely water-tight and had remained so, it may be assumed that the original concrete in the Gem Lake Dam would have given satisfactory service, as no deterioration appears in the buttresses. Also, the arches, where not subjected to freezing when filled with water, are not seriously affected as to structural strength, as is evidenced by compressive test samples taken in 1924.

The failure of the gunite coating to absolutely prevent moisture from getting in back of the top skin was the primary cause of the trouble. During the long season of freezing temperatures, extending from September or October to April or May, any moisture admitted would freeze and in expanding would either loosen the material at the face or tend to enlarge the pores throughout the mass. This operation repeated itself each time the weather warmed enough to thaw. After seven winters, slabs of gunite coating of all sizes had been loosened; that this was not due to faulty bond of the gunite is proved in part by the fact that often a thin layer of concrete adhered to the gunite when it was forced loose from the arch.

The quality of the concrete and the character of the bonding at construction joints has a direct bearing on the rapidity of disintegration, but it is doubtful whether any concrete would resist such freezing conditions indefi-

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* Constr. Engr., Nevada-California Power Co., and Southern Sierras Power Co., Riverside, Calif.

† Received by the Secretary, October 26, 1925.

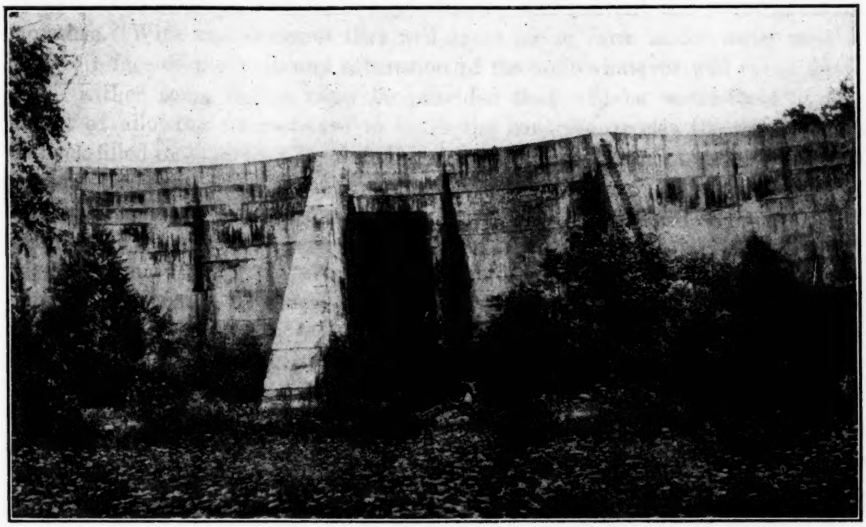

FIG. 12.—MULTIPLE-ARCH DAM IN CENTRAL NEW YORK, DOWN-STREAM FACE, SHOWING DISINTEGRATION.

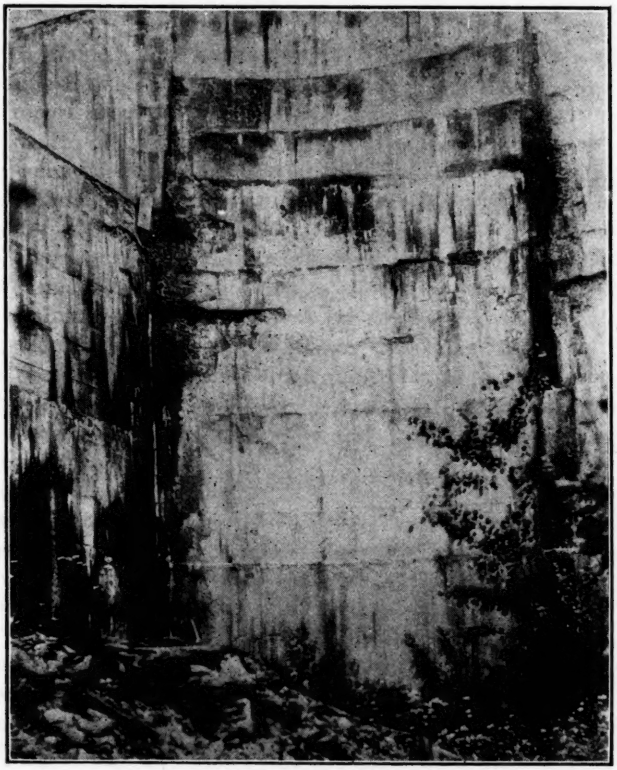

FIG. 13.—DETAIL OF DOWN-STREAM FACE, MULTIPLE-ARCH DAM (LEFT BAY IN FIG. 12), SHOWING EXTENT OF SPALLING.

nitely. It is inconceivable that any concrete will prevent the penetration of moisture. With temperatures that will cause ice to form under water on the reservoir face of the wall, any saturation of the wall whatever will cause damage. Either some facing must be provided that will be water-tight to the extent of allowing no moisture to reach the concrete or else the dam should be back-filled or otherwise insulated against cold. In the original construction a water-tight metal up-stream form would be the most satisfactory solution. It could be asphalt dipped, or otherwise treated, so that it would not rust against the concrete and it could be repainted when necessary on the up-stream face and maintained in water-tight condition at comparatively little cost. Such a form could be designed with expansion joints. In this way the concrete in the arches could be kept dry and the full strength would remain so long as the up-stream metal facing were maintained water-tight. It would be impracticable to carry the metal form below the ground line, but at this point it would be a simple matter to fully protect it by back-fill and still leave the structure open for observation.

It is possible that metal is still the best protection for the Gem Lake Dam, but it is much more difficult to place than it would have been in the original construction. Some substance that has the qualities of adhesion and elasticity and can be sprayed or brushed may solve the difficulty; field tests are now being made with certain bituminous compounds to determine the feasibility of their use.

ALFRED D. FLINN,* M. AM. SOC. C. E.—There is danger in over-emphasizing one notable failure. It is creditable to engineers who have knowledge of failures to report them for the benefit of the profession; but it would also be of service if the Society, through a small committee of inquiry, or an existing committee, would set down against this failure the record of the thin concrete dams under as great or even greater exposure that have stood successfully for years. There are many such dams in the United States. The importance of presenting such countervailing information is not slight. Engineering Foundation might be asked to collect the data for the Society.

Dams are among the structures that are supervised by some States. Some supervising officials are not well informed about the theory or the practice of the construction of dams. Their responsibility is to make sure that structures which they approve do not fail. The cost put on the owner, or the limitations on the engineers in charge of design and construction, are only secondary to the officials.

Endeavors of American engineers to build successful dams at moderate cost are being watched closely by engineers in other countries. Recent correspondence with an Italian engineer indicates that some American experiences have been misinterpreted. He has been strongly opposing the use in his country of any thin dams. American engineers owe it to themselves to answer the questions which have been raised. These questions may be grouped under the heads: Sufficiency of design, suitability of materials, efficiency of construction methods, and adequacy of maintenance.

For the case in hand: What is known about the materials put into the

* Secy., United Eng. Society, and Director, Eng. Foundation, New York, N. Y.

concrete which failed? What were the chemical and physical characters of the water, cement, sand, stone, and gravel? What attention was given to the proper placing and curing of this concrete for the particular purpose and conditions of this dam?

In some instances is not concrete being used with about the degree of intelligence that would use window-weight iron or alloy steel indifferently as materials for tools for cutting metal? In other words, is not concrete too often designed without sufficient regard for the purposes and conditions of the structures into which it is to be put, and with still less discrimination in supervising its mixing, placing, and curing? Concrete for simple foundations, or for structures which will be wholly protected from the weather or active destroying agents, may be different from concrete suitable for thin hydraulic structures under pressure and exposed to severe weather. This fact is so obvious that slowness in carrying it to the necessary limit in practical operations is hard to understand. Probably the tardiness to realize the relation between cause and effect has been due partly to the fact that the results of some wrongdoing in concrete construction do not appear for several years and partly to the fact that the men who design and construct often do not see their structures again and, therefore, do not learn the lessons which develop during years of maintenance. However, notable progress has been made.

In engineering textbooks, reports, journals, and specifications it is commonly implied that Portland cement is uniform; that cement from any source may be used just like cement from any other source; but experience in testing and using cement shows that, instead of being uniform, cement is a various material and that some cements are not well suited to certain special services, for example, hydraulic structures. It is encouraging to know that a fundamental study of hydraulic cements has recently been undertaken at the U. S. Bureau of Standards with the co-operation of the Portland Cement Association, and that a number of engineers have been giving thought to this branch of the subject. In this connection reference should be made to the paper by William G. Atwood and A. A. Johnson, Members, Am. Soc. C. E., entitled "Disintegration of Cement in Sea Water".*

Engineering Foundation is aiding the Committee on Arch Dam Investigation, made up of members of the Society. Its experiments on the Stevenson Creek Test Dam, near Fresno, Calif., and on concrete used in that dam, and its observations on existing and proposed dams, should add knowledge that would be helpful to engineers designing and constructing dams of several types. This project has already been described.†

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 204.

† *Proceedings*, Am. Soc. C. E., October, 1925, Society Affairs, p. 340.

RELATION OF THE OHIO RIVER AND ITS TRIBUTARIES TO TRANSPORTATION IN THE UNITED STATES

A SYMPOSIUM

Discussion*

By Messrs. Harry Taylor and W. M. Black.

Harry Taylor,† M. Am. Soc. C. E.—The speaker believes that this discussion and the interest which the Society has shown in this particular question will be of great benefit to the country in general.

There are two factions in the United States that look at river improvement from two different points of view: One thinks that if there were a little more water all their troubles would be over; the other that the rivers are no good anyway, that no matter what is done to them, they will be of no benefit.

The improvements to the Ohio will go quite a long way toward answering the question. It has been stated‡ that if the matter of cost were being considered at the present time, it is doubtful whether the work would be undertaken. Whether that is so, or not, the speaker does not know, but it is perfectly safe to say that if there is no increase in the present business on the river, no one would think of spending the amount of money required to improve it; but those who have studied the problem do not believe that the present business represents by any means the ultimate amount that will be carried by the river. That may be shown pretty conclusively by looking at what some of the business institutions on the river are doing, such as the steel corporations at Pittsburgh, Pa. Jones and Laughlin and the Carnegie Steel Company are not firms that spend large sums of money in unprofitable business ventures as a rule; and they are spending large sums in the development of floating plants. During the season of the year when there is sufficient water in the Ohio, these companies carry large shipments down the river into the Mississippi, for delivery at St. Louis, Memphis, and New Orleans, and even for shipment abroad. The speaker has recently noted a statement in the press that one of the largest shipments of steel products that had ever left Pittsburgh was recently taken down the Ohio and delivered at Memphis in eleven days from the time of departure. That is not bad time for a railroad, so that the element of slow movement can be omitted.

In making recent allotments for the river improvements, the Ohio River has been given very liberal consideration by the Federal Government. The

* Discussion continued from November, 1925, *Proceedings*.

† Maj.-Gen., U. S. A.; Chf. of Engrs., U. S. A., Washington, D. C.

‡ *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1672.

engineers engaged on this work would like to see the improvements finished, and, with ordinary luck, it is hoped to be able to finish them in the fall of 1929.

That there has not been a very large through movement of freight on the river to date is natural, when it is considered that during much of the year there are only 2 or 3 ft. of water at the lower end, making navigation of the river impracticable. With the improvement completed and with the 9 ft. depth throughout, that handicap will be overcome.

Something of the future of the Ohio River can be predicted from what is happening on the Mississippi. As is probably known, the Government is operating a barge line between St. Louis and New Orleans. The equipment was built during the World War, and a large part of it is not adapted to the service. The towboats are too large and heavy, with too deep draft for the river between Cairo and St. Louis; some of the barges were not exactly suited to the business, and the towboats were picked up where the management could find them; but notwithstanding these physical handicaps, that barge line is developing a large and profitable business.

In addition to the physical handicap that the line had to overcome, the lack of interchange of freight with the railroads had to be met. At present, the barge line has traffic arrangements with 165 railroads, so that it covers practically the entire country. It has been of great benefit to the farms in Kansas. In a pamphlet recently published by the National Rivers and Harbors Congress for distribution among the agriculturists is quoted a statement from an address before the Twentieth Convention of the National Rivers and Harbors Congress held at Washington, D. C., on December 10, 1924, by the Hon. F. E. Frizell, of Larned, Kans., who is a farmer, a cattle raiser, and a member of the State Senate. He said:

"We have learned that the Federal Barge Line between St. Louis and the Gulf has made a substantial reduction in the freight rate on wheat shipped from Kansas for export. The rate from Kansas to New Orleans, all rail, is 30½ cents per 100 lb., while the rail-and-barge rate is 25 cents, a saving of 5½ cents per 100 lb.—something over 3 cents per bushel saved by shipping our wheat from Kansas to either St. Louis or Cairo to reach the northern terminus of the Federal Barge Line. I have it on good authority, in fact from experts, that with the Missouri River properly improved to Kansas City, Kansas, a saving of another 3 cents per bushel could be made to the farmers of Kansas.

"Since the farmer, in effect, pays the freight on all of his wheat to Liverpool (since that market fixes the price of all the wheat whether the wheat is actually exported or not), when we compute the value of Missouri River navigation to the Kansas wheat farmer, we cannot merely calculate the saving on the entire Kansas crop, because, if you raise the price of the export wheat 3 cents per bushel by reducing the cost of the freight to Liverpool by 3 cents, you automatically raise the price of all the wheat crop in Kansas.

"A reduction of 3 cents per bushel in the freight to New Orleans means a saving of $4 500 000 on the 1924 Kansas wheat crop of 150 000 000 bushels. With the Missouri River improved, a net saving of 6 cents per bushel could be made, which would mean a saving of $9 000 000 to Kansas wheat farmers for the year 1924.

"It is estimated that the State of Kansas has raised and sent to the markets of the world an average of 100 000 000 bushels of wheat each year since the plan to improve the Missouri River was first made fourteen years ago, in

1910. That would make the stupendous total of 1 400 000 000 bushels that has been shipped out by Kansas wheat farmers during the fourteen years we have been patiently waiting for Congress to furnish the money to improve this great water highway. It would have cost not to exceed $20 000 000 to improve the river to a permanent depth of 6 ft., but it has cost the wheat farmers of Kansas more than $40 000 000 not to improve the Missouri River as was recommended by the Engineers Corps."

In the fall of 1920 crops were good, the demand was urgent, and prices were high, but the railroads could not furnish cars and the crops remained unmarketed until the demand had been supplied from other sources and prices had been cut in half.

That was a case where, while the river probably could not carry all the wheat that was offered, it might have carried an appreciable portion of it. The small relative amount of commerce carried by the Ohio compared to that carried by the railroads was also referred to by Mr. Frizell. The Panama Canal carries a relatively small commerce as compared with the trans-continental railroads, but, as is well known, the Panama Canal has had a marked effect on transportation conditions in the United States.

The rivers, the speaker believes, should be made a part of a great transportation system; they must fit in as parts of the system. They cannot operate by themselves; they can only reach the cities on their banks. Operating in connection with the railroads, they can reach practically every part of the United States, as the barge line is doing. As to the barge line, it may be stated that, while it is a Government-owned transportation system at present, operated under the authority of Congress, the endeavor is to establish a successful line and then sell it to private transportation interests. The Government has not authorized that line to continue indefinitely. The speaker doubts whether any private corporation could have financed it through its struggle for existence to build up a business. It appears to be reaching a condition, however, where its success is assured, with more business offered to it than it can possibly handle; and it has these interchange rates established throughout the country.

Reference has also been made to a waterway connecting the Great Lakes and the ocean.* The speaker does not know what particular waterway was referred to, whether it was the St. Lawrence Waterway, the one across the State of New York, or that across the State of Ohio. They have all been under consideration.

Several years ago, Congress authorized an investigation of the practicability and advisability of building a barge canal to connect the Ohio River and the Great Lakes. This subject is still under investigation. Recently, Congress also authorized the investigation of the question of the construction of a ship canal across the State of New York. Little progress has been made on this investigation to date, but it may be said that before the existing Barge Canal across the State of New York was constructed the State authorities went into the question as to whether the canal to be constructed should be a barge

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1673.

canal or a ship canal. The ship canal project was rejected as too expensive. Since that time, the cost has increased very greatly.

The St. Lawrence River is being intensively studied unofficially at the present time by Herbert Hoover, Hon. M. Am. Soc. C. E. Probably that waterway will some day be improved. When or how cannot be stated, but on the St. Lawrence River to-day there is being wasted about 4 000 000 h.p., which waste cannot be allowed to go on indefinitely. The power will have to be developed. With the increasing price of coal and the greater need for power, the time will come when the demand for that development will be so great that it cannot be resisted. It is a complicated question, because it involves international problems; it also involves very difficult, but not unsolvable, engineering problems. When it is developed for water power, it will require a comparatively small additional expenditure for navigation. What kind of navigation will take place is a problem that only the future can answer. Something of the same condition exists on the Tennessee River, which is one of the tributaries of the Ohio. The U. S. Corps of Engineers is making a survey at present of the Tennessee River water-shed for navigation and power development. The indications are that between 2 000 000 and 4 000 000 h.p. can be profitably developed. That will involve a system of dams which will practically canalize the river and will make navigation possible with comparatively little additional expense. With the demand for power in the South, which is increasing very rapidly and which is outgrowing the capacity of the present companies, it is only a question of time when that development will be made. Whatever navigation there will be, therefore, on that river, will be added to that on the Ohio. Taking into consideration the development of the Tennessee and of the Ohio, the growth of the Mississippi traffic and the rapidly increasing interchange of rail and water freight, undoubtedly in time the improvement of the Ohio River will justify itself.

Upon what takes place on the Ohio and the Tennessee Rivers will depend very largely the development on other rivers. Three or four years ago, the streams which the U. S. Engineer Corps had under consideration were divided into rivers of first importance and rivers of secondary importance. The Ohio, of course, is included under rivers of first importance. Of an appropriation of about $50 000 000, $80 000 was allotted to the rivers of secondary importance, showing a practical standstill with respect to the smaller rivers. Actually, there was a sum larger than $80 000—it amounted to about $700 000 or $800 000 for all the rivers in the country for maintenance, that is, for keeping them *in statu quo,* so that if they were subsequently improved there would be no great difficulties. If all work should stop at the present time, there would be an accumulation of sand-bars, which would eventually cost a great deal of money to remove.

In connection with the smaller rivers there was formerly a great deal of politics which controlled largely the appropriations and improvements. In 1902, conditions reached such a stage that Congress directed that a board known as the River and Harbor Board, be authorized to pass on all improvements. Since then there has been practically no improvement authorized by

Congress that has not been recommended by that Board. About 70% of all the projects reported on by the Board have been reported unfavorably. In the last five years, Congress has adopted the method of making lump sum appropriations instead of itemized appropriations. Formerly Congress directed where the money should be spent; it is now appropriated as a lump sum available for allotment by the Chief of Engineers, so that if anything is spent on worthless engineering projects, the Chief of Engineers is responsible.

The Muskingum River was improved many years ago by the State of Ohio, and turned over to the United States to carry. These improvements cost a considerable sum of money. The Big Sandy River was improved by the Government, as was also the Cumberland River, the first appropriation for the latter stream having been made in 1832. While the ton-miles of commerce on those streams is small, the only money being spent on them at present is that necessary to keep up the improvements already made. Those improvements consist of locks and dams, at each of which there is a small crew. The total expenditures on the Big Sandy amount to a few thousand dollars only per annum. The same is true of the Muskingum. On the Cumberland River System the last remaining dam below Nashville was completed in 1924; the expenditures on that stream will also be very small hereafter unless future developments justify further work above Nashville. So that, to say that what is now being spent or that the improvements now being made are for political reasons, is misleading. It was formerly so, but not now.

W. M. Black,* M. Am. Soc. C. E. (by letter).†—Colonel Kutz's valuable paper‡ should be considered together with the papers§ by H. B. Luther and Frank H. Alfred, Members, Am. Soc. C. E., relating to the same subject, and the discussion on the writer's paper "Waterway and Railway Equivalents".‖ The diversity in the opinions expressed by the railway officials on the one hand and by the advocates of water transportation on the other is marked and, to the mind of the writer, unfortunate.

The question of transportation is vital to the prosperity of the people. The solution of the many intricate problems involved in its development should be based on the general welfare rather than on the interests of any industry, locality, or section.

The railway is an essential factor of modern economic life. No other agency of transportation as yet devised can to-day do its work. Yet when it is considered that in times past the transportation agency of primary importance has been, successively, the trail, the waterway, and the railway, and that, to-day, the motor truck and the airplane are successfully displacing the railway car for certain transportation uses, it seems possible that some day, the railway in turn may be relegated to a secondary place.

It is a platitude to state that each transportation agency has its own truly economic place and work, and that the nation is entitled to have its trans-

* Maj.-Gen., U. S. A. (*Retired*); Cons. Engr. (Black, McKenney & Stewart), Washington, D. C.

† Received by the Secretary, October 28, 1925.

‡ *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1642.

§ *Loc. cit.*, pp. 1659 and 1663.

‖ *Transactions*, Am. Soc. C. E., Vol. 88 (1925), pp. 553–577.

portation bills reduced to a minimum by having each agency perform its economic share of the work. Such is not the case to-day; and is it not because a destructive competition exists between the owners and managers of the various agencies?

Colonel Kutz's Table 9,* giving the fixed overhead charges per ton-mile for transportation on the rivers of the Ohio Valley, shows that with the limited use made of these waterways to-day, not only are some of them uneconomical agencies now, but, also, that their time of usefulness seems past and that the propriety of the discontinuance of operations on them should be considered, in conformity with the policy already followed in the cases of some other waterways, as well as of certain branch railways. The other tables in the papers, together with data given in the discussion, show some of the reasons why the tonnage of the waterways is so limited.

Carriage by water, mile for mile, is inherently less costly than carriage by rail. There is less likelihood of breakage, and, within limits, actually quicker delivery. The advent of the motor truck, which has curtailed short-haul railway service, has but increased the value of the waterways by broadening the territory which they can serve.

Why, then, are not the waterways in greater use in their own sphere of usefulness? The writer believes it is because of the improper conception of the use of railways which has produced in the course of years that cumbersome and unstable structure, the railway freight rate tariff.

This tariff is full of inconsistencies and full of discriminations. It is so complicated that only an expert can use it to discover a rate. In 1921, the writer found it necessary to ascertain the rates on certain ordinary package freight in five cases of movement between large cities of the United States. It required 9¾ hours for an expert of the Interstate Commerce Commission to pick out the rates from the mass of matter he had to go over!

The railway freight rate tariff is a subject concerning which railway officials are very sensitive, both as to freight classifications and freight rates. It is defended by many strong arguments, and under existing conditions some of these arguments are unanswerable. It is, however, an inheritance from the days when railways had to struggle for existence as radical and impracticable innovations, and from the later days of destructive competition among the rival roads. In those days public service in return for privileges granted was a secondary thought. The policy of the railway managements was based instead on the desire of gain for themselves. "The public be damned" slogan reflects this thought.

The railway freight tariff of to-day is an anomaly in the scheme of government of the United States. It is not in accord with the spirit of the Constitution that any body of men, other than duly elected legislative representatives, should be empowered to tax one community of the United States for the benefit of another, to enforce what is really a protective tariff in favor of one city or locality as against another, or to foster one class of products at the expense of another. This, however, is done by the existing railway

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1656.

freight rate and freight classification system. Proposals for any radical changes are strongly resisted by those now enjoying the favors conferred.

The entire structure is artificial and is almost as unstable as a house of cards. Change it anywhere and there is danger of a complete collapse. Is this a healthy condition for the principal agency of the basis of the nation's welfare—transportation?

In order to obtain its transportation at the lowest cost, the nation requires a co-ordinated transportation system, in which each agency—rail, water, highway, and air—shall perform its economic share of the work. Referring to the claim that "the development of inland waterways will relieve the railways of a large part of their freight traffic that is carried at low rates", Mr. C. H. Markham states:* "The truth about the latter argument is that almost any American railroad would starve to death if its heavy freight business were taken away from it, because that is the freight that pays best."

If this freight can be carried more cheaply by another agency, this is not an entirely fair argument from the standpoint of the general welfare. No sane person can wish the railroads to be other than prosperous. If the full use of waterways would, in truth, kill the railways, or even seriously injure their prosperity, some means of providing an adequate income for them must be furnished.

Since transportation is a fundamental element of commerce, it should be organized and managed on the most economic business basis. Should not a transportation company be prepared and able to utilize the most economical and advantageous class or classes of carrier for any specific cargo offered for a given destination? If each auxiliary transportation agency were used for the work it can do best, and the returns from all placed in a common fund, not only would the cost of transportation be reduced, but also the destructive competition, with its resulting artificial tariffs would disappear. A transportation company should control and utilize the operation of carriers by railway, waterway, highway, and airway along a given line of traffic movement.

It is not fully apprehended how really artificial are the present-day tariff rates, and how little the relative rates are based on the relative costs of transportation. In other branches of industry the prices asked for products or for service rendered are based on the cost of production or of the service. For the railways, the cost of the service is known in the aggregate only. The relative cost of the transportation, including the necessary care, of the various classifications of freight is unknown. Similarly, the relative costs of the line haul and of the terminal services have not been determined, although the terminal service charges are "absorbed" in the freight rate charges.

Consequently, when railway revenues become insufficient there is no recourse other than that of an arbitrary increase of one or more of the specific rates. An example of the peculiar attitude taken toward freight rates is afforded by the recent action of the Interstate Commerce Commission in ordering an arbitrary decrease in the rate on a certain class of coal between

* *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 555.

the West Virginia mines and New England territory. Is it safe or sane to keep so vital a thing as the National transportation on such an unstable basis?

At one time, competition between railways was deemed necessary in order to secure an economic service. To-day, it is recognized that each railway line enjoys a natural monopoly for certain territory, and, further, that a railway affords a public utility service and that it, as other analogous utilities, serves the public best as a regulated monopoly. Is there any valid reason why the service should not be extended to cover other classes of common carriers, and thus to serve the public to greater advantage?

Water competitive railway rates are allowed by the Interstate Commerce Commission on the grounds that such lessened rates are necessary in order to permit the railways to meet the competition of the waterways. Those water competitive rates necessarily imply higher rates for interior points, and, therefore, unequal treatment for similar service. If it be the case that freight can be carried by water more cheaply than by rail (which is denied by some railway experts), and if transportation companies were authorized to operate inland waterway lines, the reasons for the water competitive railway rates would disappear, with the resulting discriminations against interior points. If, as is contended, water transportation is not cheaper than rail, why the water competitive rates?

It is impossible to consider the subject of interior transportation without being confronted with the basic subject of the railway freight rates and freight classifications. Without a radical change in these, a harmonious economic system of interior transportation cannot be devised. The subject is so complex and so little is known as to actual costs, that a rational solution of the question cannot be had without an investigation and study which will be long and costly. Is not the question worthy of the time and expense?

Even with a solution presented, changes would have to be made slowly, step by step. In the meantime, cannot a start toward reform be made by permitting the railway companies, under due regulation, to take over and operate interior waterways lines within their respective zones, and the abolition of water competitive railway rates?

REPORT OF THE JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

Discussion*

By J. A. Kitts, Assoc. M. Am. Soc. C. E.

J. A. Kitts,† Assoc. M. Am. Soc. C. E. (by letter).‡—The report§ of the Joint Committee on Concrete and Reinforced Concrete admits the lack of an accepted comprehensive technology of concrete manufacture. This lack is not a fault of the Committee, but is due to a peculiar situation prevailing in the concrete-producing industry.

The production of concrete should be viewed as one of the most extensive of American manufacturing industries. It is, however, an industry peculiarly without a stable centralization or organization, such as prevails in the manufacture of metals, Portland cement, and other products which have a highly developed technology. The consumer of concrete is usually the producer *pro tempore* and his interest in concrete manufacture is correspondingly a temporary one. The development of the combined practical and scientific knowledge of concrete has been naturally retarded by the want of a sustained interest in concrete research and by the lack of co-operation in, and co-ordination of, such research.

Any one can make concrete of a sort by arbitrarily mixing rock particles, hydraulic cement, and water. This fact has tended to overshadow, in the minds of the industrialist, of the builder, and of the engineer who uses concrete in his projects, the economic importance of concrete research and the development of practical and scientific processes of concrete production.

In the past, the burden of the development of a technology of concrete manufacture has been left largely to the individual resources of a few enthusiasts (who do not agree) and, consequently, the industrial science of concrete is still in the formative stage despite 100 years production of Portland cement and more than 1 000 years since the first use of concrete.

Considerations of economy in local and National development involving more than 100 000 000 cu. yd. of concrete per annum demand an intensive study of concrete. This could best be accomplished by sectional co-operative concrete

* Continued from September, 1925, *Proceedings*.

† Concrete Technologist, Berkeley, Calif.

‡ Received by the Secretary, September 15, 1925.

§ *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

research and study of local materials by producers, consumers, cities, counties, States, architects, engineers, industrialists, utilities, etc. Co-operative and co-ordinated effort of this character is one of needs of a joint committee on concrete and such assistance has been solicited by such committees in the past.

The production of concrete is a manufacturing industry peculiarly in the province of the civil engineer. With the engineer lies the burden of responsibility for the development of a technology of concrete. Engineers have brought forward many fundamentals of the physics of concrete, methods, processes, and machinery, and have individually improved concrete technology. A common fault, however, of the engineer doing concrete research is that he is prone to base his theory of concrete proportioning on the one principle discovered by himself to the exclusion of all principles discovered by others, and that he has neglected to apply the fundamental physics and mathematics in which presumably he has been trained.

A theory of concrete based on a principle of fundamental physics may be co-ordinated with all other theories of concrete based on similar principles. Authors, however, disagree and the status of the knowledge of concrete is that of disagreement. This is the condition confronting the Joint Committee.

The solution of the problem of agreement on theories of concrete proportioning depends on the application of fundamental physics in the study of these theories. Accordingly, a technology of concrete should include the following subjects:

1.—Chemical and physical properties of various hydraulic cements and cement mortars, and tests for such properties.

2.—The technology of aggregate production: Standardization of screening sizes; requirements of plant for elimination of silt, organic matter and any deleterious substance; and control tests.

3.—Petrography of aggregates; physics of aggregates and of aggregate mixtures; tests for specific gravity, density, grading, absorption, moisture, bulking on account of moisture and of loose measurement, bulking in inundation, and yield, in "aggregate" volume, of combination by "apparent" volumes; mathematics of grading of sizes, etc.,

4.—Physics and mathematics of concrete mixtures and tests: Size, grading, density, specific gravity, and absorption of the mixed aggregate; ratio of cement to mixed aggregate; yield of combination; basic water content for maximum solids in the mass; ratio of mixing water to basic water for desired workability; consistency; water to cement ratio; proportions of all materials by dry-rodded, loose-moist, actual, and inundated volume, and by weight; ratio of cement to voids in aggregate; ratio of cement to space occupied; ratio of sand to voids in rock, etc.; density, impermeability, wear, strength, and absorption of the concrete.

5.—Technology of concrete mixing, placing, curing, field control, and tests: Essential measuring devices, mechanical devices for mixing and placing, control tests, and apparatus.

6.—Specification of minimum requirements of tests, testing apparatus, inspection, etc., at aggregate plant, central laboratory, and job.

The report considers only Portland cement in Item (1); it treats of mixing and placing in Item (5) quite adequately; and it is almost totally lacking in the physics of aggregates, aggregate mixtures, and concrete mixtures.

The presumably empirical but practically arbitrary proportions, as given in Appendix XVI* will serve better to retard the development of a technology of concrete than to produce economically good concrete. Most of the mixtures are fundamentally in error in that they have a gap of sizes in the grading of the aggregate. The table† is based on the assumption that all ¾-in. to 3-in. aggregates, for example, have the same physical arrangement or properties (density, size, grading, etc.). It furnishes no basis whatever for the proportioning of three or four aggregates as used on the Exchequer Dam project.

A joint committee cannot be expected to produce a technology of concrete until fundamental physics and mathematics are utilized in concrete research as a part of construction. The use of elementary and fundamental physics in a comprehensive, systematized, and co-ordinated laboratory procedure in the testing of materials and the technical proportioning of concrete mixtures as an every-day operation of construction shows, as the writer proposes, that science and practice may be combined without conflict and with many advantages.

Arbitrary and empirical proportioning of concrete mixtures can be excused only on the grounds that it is the best that is known. It is the general practice and the report of the Joint Committee reflects this situation.

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1277.

† *Loc. cit.*, p. 1279.

THE IMPROVED VENTURI FLUME

Discussion*

By Messrs. H. B. Muckleston and E. W. Lane.

H. B. Muckleston,† M. Am. Soc. C. E. (by letter).‡—The measuring device described in this paper should hardly be called a Venturi flume, at least, as long as only one head is measured and the formula on page 1342§ is applied. Under these conditions the device is a weir with the bottom contraction suppressed.

The ordinary form of weir can hardly be called a satisfactory instrument for measuring irrigation water. Theoretically, perhaps, a weir is a very accurate device, but the theory implies a great many things which are seldom obtainable under ordinary irrigation conditions. It is not to be expected that any one device will be applicable under all conditions. Much depends on the system of charging for the water. When the charge is for the actual quantity delivered, some kind of integrating mechanism is necessary; and such a mechanism, applicable to any device which really measures the rate of flow, is apt to be very expensive. Under such conditions, a device which actually measures the quantity and delivers it in batches is cheaper. When the charge is for the irrigable area covered, whether or not actually irrigated, time-rate devices are the cheapest and the simplest. The main object, in such conditions, is to satisfy the user that he is getting his share of the supply. Much depends on whether each user must have his individual measuring-box. When water is rotated among a group, group measurement is all that is really necessary. The quantity delivered to the unit then depends on the time of flow.

Considering only time-rate devices, a satisfactory measuring instrument must meet a number of conflicting requirements. It must be cheap, nearly "fool-proof", immune to silt troubles, easily read, and fairly accurate; it should hold its accuracy under considerable abuse, and it should require a minimum of lost head. The weir meets the first and fourth requirements very well, but its virtues stop there. The Venturi flume, as first described by Mr. Cone and afterward by the author, came more nearly to meeting all the conditions than any other with which the writer was acquainted. It is possible that the new device will do even better.

* This discussion (of the paper by Ralph L. Parshall, Affiliate, Am. Soc. C. E., published in September, 1925, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Vancouver, B. C., Canada.

‡ Received by the Secretary, October 10, 1925.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions.

One of the main objections to the weir was the loss of head which it involved. This might be several inches or even a foot or more. One consequence was that the user developed a strong tendency to back up the water and drown the weir. The original Venturi flume was fairly accurate even when deeply submerged, and the use of a plug, as described by Mr. H. K. Smith, extended the accuracy over a wider range. The new device gives promise of sufficient accuracy, and it should be nearly proof against abuse; but it should be noted that, as long as it must work under the condition of free flow, the possibility of the user drowning the flume is present. The indications from a single head measurement will then be inaccurate, possibly greatly inaccurate. The device promises to be a cheap one.

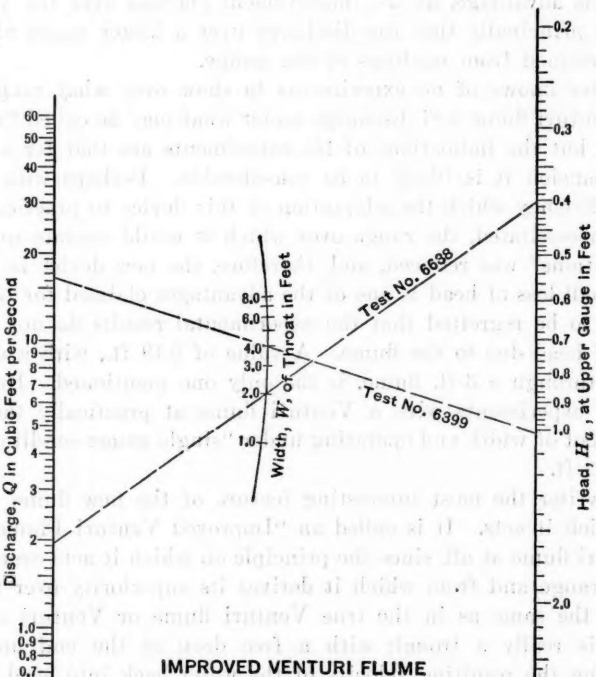

FIG. 5.

As long as other factors are so indefinite, third decimal accuracy is not required in measuring devices. Under any conditions, 6% should be close enough; less than 3% would be splitting hairs.

The rather formidable looking formula on page 1342* is easily plotted as an alignment nomograph, Fig. 5. The lines for Q and H_a are simple loga-

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions.

rithmic scales to any convenient pattern. The line for W can be established by cross-intersections, using the calculated discharges from Table 2.* No calculations are needed.

E. W. LANE,† M. AM. SOC. C. E. (by letter).‡—In his paper, "Experiments on the Flow of Water Through Contractions in an Open Channel",§ the writer pointed out that the experiments for developing the Venturi flume for measurement of irrigation water dealt only with conditions of discharge under which readings of both up-stream and throat gauges were necessary to determine discharges, and that for a large range of conditions the discharge could be determined from readings on the up-stream gauge only, the advantage of which had been overlooked. The flume described by the author seems to be a recognition of this advantage, as the improvement claimed over the Venturi type seems to be principally that the discharge over a larger range of conditions can be determined from readings of one gauge.

The writer knows of no experiments to show over what range of conditions the Venturi flume will discharge under what may be called "single gauge conditions", but the indications of his experiments are that for a flume with gradual expansion it is likely to be considerable. Perhaps with the loss of hydraulic efficiency which the adaptation of this device to practical irrigation conditions necessitated, the range over which it would operate under "single gauge conditions" was reduced, and, therefore, the new device is better.

Since small loss of head is one of the advantages claimed for the improved flume, it is to be regretted that the experimental results do not include the total loss of head due to the flume. A value of 0.19 ft., with a discharge of 9.48 sec-ft. through a 3-ft. flume, is the only one mentioned. Run No. 3 of the writer's experiments with a Venturi flume at practically the same discharge per foot of width and operating under "single gauge conditions" showed a loss of 0.12 ft.

To the writer the most interesting feature of the new flume is the principle on which it acts. It is called an "Improved Venturi Flume", but it is not a Venturi flume at all, since the principle on which it acts over the greater part of its range and from which it derives its superiority over the original type is not the same as in the true Venturi flume or Venturi meter. The new flume is really a trough with a free drop at the end and a device for converting the resulting velocity of the water back into head. This conversion is accomplished by causing the water to flow up an inclined plane, the velocity being expended in actually lifting the mass of water to a higher elevation. There is nothing new about this in principle, but the practical application is rare. The trough part of the flume is not essentially different from a type of measuring device developed just before the Venturi flume in which the water flowed through a trough with a free, or nearly free, fall at the lower end and had a discharge varying practically as the three-halves power of the head. The effect of the converging sides of the trough section

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1348.

†Detroit, Mich.

‡Received by the Secretary, November 7, 1925.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 1201.

seems to be to increase the velocity through the narrow section, thus insuring, over a greater range, velocities higher than the critical value, and, therefore, a flow through the opening independent of back-water conditions. Although the author does not seem to have appreciated that a new principle is involved there is no reason why this flume may not be named for the originator and the name Venturi dropped.

The term, "free flow", as used by the author is confusing if not incorrect. Any condition in which water at the crest of the weir is flowing at a higher than critical velocity is a condition of free flow, since the flow is independent of the tail-water level. This may occur with the water levels indicated by the throat gauge either higher or lower than the crest of the weir. The author seems to recognize this in his title to Fig. 3* "Free-Flow Discharge, 6-Foot Venturi Flume, with Approximately 75 Per Cent. Submergence", which is obviously contradictory according to his definitions of "free flow" and "submerged flow".

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1345.

SIDE CHANNEL SPILLWAYS: HYDRAULIC THEORY, ECONOMIC FACTORS, AND EXPERIMENTAL DETERMINATION OF LOSSES

Discussion*

By Messrs. C. H. Howell and H. B. Muckleston.

C. H. Howell,† M. Am. Soc. C. E. (by letter).‡—The writer believes Mr. Hinds has successfully solved a complicated hydraulic problem of considerable economic importance and scientific interest. To design a spillway properly is often more difficult than to design a dam. This is especially true with very large discharges. A rational method of analysis and design such as this paper presents will be valuable to engineers.

The confirmation of the theory by the Bellevue and Arrowrock experiments will increase the confidence of those using it. As the author states, this type of spillway is rather inefficient hydraulically and is best suited to moderate discharges. The writer's experience leads him to believe that large discharges, exceeding 20 000 to 25 000 sec-ft. can be handled much more economically by spillways with gates placed normal to the channel. The economy increases rapidly as the discharge increases above 25 000 sec-ft. With a gate or gates placed at the upper end of and normal to the channel, the velocity at the gate can be either maintained or increased by a proper slope. The large losses due to impact, inherent in the side channel spillway, are thus avoided.

The side channel spillway, however, lends itself well to the installation of segmental drum crests. These crests, as developed by the U. S. Bureau of Reclamation, are positive in their automatic action. With earth or loose rock dams a large part of the total spillway discharge should have automatic control. The value of the automatic feature of the drum crests may sometimes offset the inefficiency of the side channel spillway.

In designing the Guernsey Dam (now under construction), on the North Platte River, the Bureau of Reclamation was confronted with the problem of discharging 80 000 sec-ft. past a gravel-fill dam about 100 ft. high. Automatic control for 30 000 sec-ft. was assumed. This is about the maximum discharge in 34 years of record. The side channel type of spillway was tried in various forms and abandoned on account of its excessive cost. The final designs contain some unusual features. The discharge of 30 000 sec-ft. is controlled by

* This discussion (of the paper by Julian Hinds, M. Am. Soc. C. E., published in September, 1925, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Engr., U. S. Bureau of Reclamation, Denver, Colo.

‡ Received by the Secretary, October 9, 1925.

two automatic segmental steel drum crests, each 64 ft. long and 14.5 ft. high. These drums discharge into a vertical circular shaft 31 ft. in diameter which, in turn, discharges into a diversion tunnel 30 ft. in diameter. The method proposed by Mr. Hinds was used in designing this structure. This spillway is on the south side of the river. On the north side an open, concrete-lined channel, 34 ft. wide on the bottom and with ½ to 1 slopes, is controlled with one gate which is 50 ft. square in the clear and is the largest in the United States. It is exceeded materially in size, however, by some in Europe. This spillway will pass 50 000 sec-ft. without a raise in the reservoir water surface, and about 70 000 sec-ft. before the dam is overtopped.

H. B. Muckleston,* M. Am. Soc. C. E. (by letter).†—The author has not pointed out the great influence which spiral flow in the channel may have on the magnitude of the swell ratio.

The "peculiar air condition" noted on page 1381‡ was to be anticipated. Owing to the manner in which the increment water is delivered to the channel, the water in the channel tends to flow in spiral stream lines; and any entrained air bubbles, being lighter than the water, naturally seek the region of least centrifugal force, like the oil in a rifled pipe line. Some of the air bubbles are constantly escaping from the free surface; but, as more air is being added all along the line, the total air in suspension should remain sensibly constant or even increase somewhat. As soon as the air supply ceases, the "rope" dissipates quickly.

The "rope" is merely visible evidence of a condition which exists all through the mass of water in the channel so long as the flow is spiral. Below the center of the spiral the centripetal tendency due to spiral flow is assisted by the tendency of the bubbles to float; above the center, the two tendencies are opposed, and there will be a region where the resultant tendency is zero. Inside this region the resultant tendency is toward the center. Just as large bubbles rise faster in still water than small ones, so the large bubbles, in the case of spiral flow, find their way more quickly to the center, if inside the critical region, or escape from the free surface, if outside it, than small ones. It is probable that the critical region is farther from the center for large bubbles than for small ones; but, in any event, a condition of spiral flow, by retaining the larger bubbles which would otherwise escape quickly, is favorable to a high content of entrained air and a consequent high swell ratio.

It may be doubted if the effects of spiral flow can be calculated. They are likely to be the greater as the average width approaches the depth and the side slopes become steep. A wide shallow channel, besides being unfavorable to spiral flow, has a greater surface area from which air can escape. If the effects should prove objectionable in a constructed channel, they can be mitigated by building a few shallow ribs on the floor and far wall of the trough parallel to its axis. These ribs will also help in reducing the surges up the far wall.

The author's mathematical treatment of the problem is very neat and the resulting formulas will probably be ratified by experience as closely as engineers

* Cons. Engr., Vancouver, B. C., Canada.

† Received by the Secretary, October 17, 1925.

‡ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions.

have learned to expect in similar treatment of other hydraulic phenomena. The premises do not cover all the conditions, so the formulas will not predict all the results. The calculated surface profile will probably take care of the impact losses, and a few per cent. added to the calculated areas will care for the friction and other losses. The swell due to spiral flow is likely to be high but can not be calculated, and is an additional reason for a generous factor of safety in fixing the dimensions.

PERMISSIBLE CANAL VELOCITIES

Discussion*

By Messrs. H. B. Muckleston, Elbert M. Chandler, Robert S. Stockton, and R. A. Hart.

H. B. Muckleston,† M. Am. Soc. C. E. (by letter).‡—This paper attacks a problem which is in great need of serious consideration. Its principal difficulty lies in the definition of the various soils. Such designations as "sandy with admixture of clay", "clayey loam", "dark uniform clay loam", and others in Tables 5§ and 6‖ may convey varying ideas to different engineers, depending on the soils with which they have had experience. The man familiar with the loams around Winnipeg, Man., Canada, would consider the loams of Alberta as light to the point of being floury, whereas the Alberta man would consider the Winnipeg soil extraordinarily tough. Yet the designation as "dark uniform clay loam" might very well fit either material. This is not in disparagement of the authors' work, but to show the difficulties in the problem they have undertaken to solve.

The engineer who is faced with the necessity of deciding on the upper limit of permissible canal velocity for a project is confronted at the outset with the fact that he knows very little about the soils he will encounter in the excavation. No matter how numerous his test pits or borings, they can reveal little about the real character of the soils in which the canal will be dug. When the canal is entirely in an alluvial river valley there may be some assurance that the soils will be reasonably constant over an extended area, but for a canal in bench lands, particularly glacier-made lands, the character may change materially in a few hundred feet. In the writer's experience, a canal designed for a mean velocity of 2.5 ft. per sec. was badly eroded in some stretches, while in others, possibly in the next quarter-section, silting took place to such a degree as to become a nuisance. The difference in the soils was not especially noticeable on examination, but it must have been considerable in fact.

The authors are quite right in insisting that silting and scouring are phenomena of entirely different natures and following radically different laws. They involve fundamentally different processes. In the one case the force

* This discussion (of the paper by Samuel Fortier and Fred C. Scobey, Members, Am. Soc. C. E., published in September, 1925, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Vancouver, B. C., Canada.

‡ Received by the Secretary, October 6, 1925.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, pp. 1409–1410.

‖ *Loc. cit.*, p. 1411.

opposing is gravity; in the other, it is cohesion or friction. In well-packed, well-graded gravel, the resisting force is almost entirely friction assisted in some cases by the weight of the larger particles helping to hold down the smaller ones. In clays, or in materials containing a good proportion of clay, the resisting force is mainly cohesion. Before erosion can take place the particle must first be detached from its matrix and this can happen only as a result of unequal pressure on opposing sides. This inequality of pressures may be due to an upward current tending to lift the particle, or to a horizontal current tending to roll it along the bottom. If the surface is smooth and impervious, the opportunity for the disturbing currents to act on small particles is limited and a high mean velocity is permissible without scour. This explains the high resistance offered to scour by nicely slicked bed and banks. It also explains why alkali soils erode so easily.

Almost any material will erode if the water carries in suspension particles of considerable size. These particles by their momentum can detach from the bottom other good-sized particles which would not be detached by the water alone.

In discussing the quotation from the late P. J. Flynn, M. Am. Soc. C. E., the authors have mistaken his meaning in the reference to the formula of Dubuat.* Mr. Flynn was referring to Dubuat's formula for the mean velocity of water in open channels and not to his formula for permissible velocity. The former was a "horrible" expression used in India until it was superseded by Kutter's formula.

The velocities recommended in Table 6 will be accepted by most irrigation engineers, subject to their own interpretation of the terminologies.

ELBERT M. CHANDLER,† M. AM. SOC. C. E. (by letter).‡—The collection of records of experience and observation in this paper will be very useful to canal designers. The paper will undoubtedly serve to call attention to and emphasize the desirability of the use of maximum velocities consistent with the physical conditions to be met in any given case, and particularly that, under normal circumstances, the limiting factor in fixing a canal velocity should be the elimination of erosion of a seasoned canal and not the minimum velocity required to prevent the deposit of silt, as has been the case in many instances.

Of course, each particular situation must be analyzed and the conditions met as they arise. For example, in the case of canals diverting water from the Snake River and carrying a moderate but not harmful quantity of silt, circumstances could easily require, not the maximum or limiting permissible velocity, but some velocity less than the maximum that will permit of a continuous slight deposit of silt, such silt deposits being useful and necessary to reduce seepage losses.

There are certain kinds of aquatic growth that thrive only in water having a very high velocity. The writer has noted the odd situation of no aquatic

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1407.
† Civ. Engr., Olympia, Wash.
‡ Received by the Secretary, October 13, 1925.

growth in the canal, but in an adjacent regulating spillway having a continuous velocity well over 10 ft. per sec., long strings of such growth.

Another factor to bear in mind is that many circumstances will justify in initial construction a concrete lined canal, not only because of the reduction of seepage losses, but of the decrease in the cost of excavation owing to the use of a smaller sectional area. This possibility should always be studied when giving consideration to grades and maximum velocities.

It may not be amiss to call attention to the life work of one of the authors, Dr. Fortier. For more than forty years, he has studied, investigated, and worked to solve the many difficult problems incident to irrigation development. He was the first to emphasize that the work of construction and operation of irrigation works was purely incidental and secondary to the ultimate result—the success of the man on the land. Twenty years ago he was a pioneer in working to improve the methods of irrigation and the opportunities of the irrigation farmer. His rôle has not been spectacular, but the actual worth of his services to the irrigated West has been exceeded by none.

ROBERT S. STOCKTON,* M. AM. SOC. C. E. (by letter).†—This paper emphasizes certain matters in connection with permissible canal velocities that have come under the observation of experienced operating engineers in this field, but that have not been fully discussed in textbooks. This subject is one into which many factors enter, and for which the collection and tabulation of observed field data at a large number of locations as given in the paper are particularly valuable.

It has been noted that a canal carrying water at a velocity near the eroding point on a straight course will actually erode on the curves. It may be desirable to use such a velocity and protect the curves by rip-rap or by superelevation of the bottom to correspond with the increased velocity on the outside of the curve. This latter method is limited in its application but has not been used as much as it should have been.

As a practical matter of economical operation an engineer should be sure that the velocities in small canals and distributing ditches are high enough; on the other hand, in designing large canals he should perhaps lean the other way and be sure the velocity is not too high, if they carry mostly clear water through material where seepage may be an important matter. For very muddy waters, for a low construction cost, and for preventing the growth of water plants, the highest permissible velocity or the highest feasible velocity would be indicated. At many points the soil is in layers and the softest must, more or less, govern the velocity. As pointed out‡ by Fred D. Pyle, M. Am. Soc. C. E., it is usually much easier to remedy a high velocity than to increase a flat grade in a canal. The lighter the soil, the flatter the side slopes must be for stability.

The canals on the Western Section Irrigation Project of the Canadian Pacific Railway, Department of Natural Resources, have been in use from

* Supt. of Operation and Maintenance, Western Section, Canadian Pacific Railway Co. Irrigation Block, Strathmore, Alberta, Canada.

† Received by the Secretary, October 13, 1925.

‡ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1405.

15 to 20 years. The soil varies from sand to heavy clay, but in most places contains no gravel and might be described as varying between a light sandy loam and a clay loam, all characteristic of the prairie soils east of the Rocky Mountains.

The Main "A" Canal of this System is through an average clay loam soil and is a good example of a large canal in good condition after twenty years of use. The constructed slopes were 1 on 2 and for long distances are in almost perfect condition, except for occasional gullies where storm water has entered from the side. Of course, the section is modified slightly and approaches the parabolic form so pronounced on the canals and ditches in the softer loam soils. The calculated velocity for a depth of 10 ft. and a discharge of 2 048 sec-ft., is 3.2 ft. per sec., with $n = 0.025$. The discharge of the canal has not exceeded about 1 500 sec-ft. and for most of each season runs at half capacity or less, so that conditions are favorable for maintaining the canal section. The Bow River water turned into the canal carries silt for about a month at the high-water period. The grade of the canal is 0.02% and the typical meter measurements at Chestermere Station are as given in Table 7.

TABLE 7.—TYPICAL METER MEASUREMENTS OF MAIN "A" CANAL AT CHESTERMERE STATION.

Width, in feet.	Area of cross-section, in square feet.	Mean velocity, in feet per second.	Discharge, in second-feet.	Depth of water, in feet.	Calculated roughness coefficient.
52.0	116.0	1.25	145	2.70	$n = 0.028$
63.0	254.0	1.90	436	5.10	$n = 0.028$
72.0	459.0	2.74	1 257	7.90	$n = 0.026$

The material would come under the heading of "ordinary firm loam" and the canal would be classified as "carrying water bearing colloidal silts during a part of the season".

The condition of the canal substantiates the recommended velocity as shown in Table 6.*

The Secondary "A" Canal runs for most of its length through a rather sandy loam and follows the contour closely so that it has a large curvature. Considerable cutting requiring rip-rap work has occurred on this canal, mostly on the curves. The designed velocity varied in the sandy section from 2.5 to 2.7 ft. per sec., but experience indicates that it should have a maximum mean velocity of about 2.0 ft. per sec., for stable banks, and the material should be classed as a fine or sandy loam in Table 6. This canal takes practically clear water from Chestermere Lake but at high stages is turbid from the erosion of its own banks. The meter measurements as given in Table 8, are typical.

In general, it will be admitted that a large number of complex and variable conditions affect the design of canals and ditches as expressed by the permissible velocities of the water and are difficult to place in their true per-

* *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1411.

spective without considerable practical experience. It is of great benefit, however, to try and describe fully all the factors and discuss conditions in order to capitalize the united experience of engineers and express as clearly as possible all the principles involved.

TABLE 8.—TYPICAL METER MEASUREMENTS AT BRIDGE 16A, CAIRNHILL. (Grade, 0.03%; constructed side slopes, 1 on 2.)

Width, in feet.	Area of cross section, in square feet.	Mean velocity, in feet per second.	Discharge, in second-feet.	Depth of water, in feet.	Calculated roughness coefficient.
27.0	34.3	1.47	50	1.6	$n = 0.020$
31.0	67.0	1.90	127	2.7	$n = 0.022$
41.5	158.8	2.56	408	4.8	$n = 0.024$

It very frequently happens that the cost of construction, plans for enlargement or future canal lining, the requirement for a high velocity to prevent weed growth, or other factors, lead to the selection of different velocities from those most favorable to a condition of no erosion or silting. It is frequently the case that the material through which canals are built changes so often from one class of soil to another, that the only practical course is to meet a probable average condition.

On the Western Section Irrigation Project, it has proved wise to leave many small ditches with excess grade and to utilize the natural channels. The Operating Department has thus been able to take up the excess grade after use has developed its exact amount and determined the location of controlling structures.

The writer has had particular occasion to observe the difference between new and old canals and wishes to commend the emphasis the authors have laid on designing for the more or less permanent condition of the old canal with its compacted banks and growth of vegetation.

R. A. HART,* M. AM. SOC. C. E. (by letter).†—This paper should prove of great value not only in that the various factors relating to the question have been brought together but especially in that the difference between eroding and transporting velocities is so clearly emphasized.

An important consideration is the position of the ground-water table with respect to the water surface of the canal. As long as the latter is higher, seepage is out of the canal, and there is a tendency for the finer materials to be carried into the interstices between coarser particles, thereby permitting a silting-up process. On the other hand, if the ground-water table is higher, as frequently happens, seepage is into the canal and the whole process is reversed.

In localities having a high water-table there is likely to be an accumulation of soluble materials in the soil, in the ground-water, and even in the canal water. The nature of the soluble matter has a bearing on the case. Calcium salts aid in the flocculation of soil, forming a structure that is easily eroded, whereas sodium carbonate, for example, has a deflocculating effect.

* Drainage Engr., U. S. Dept. of Agriculture, Salt Lake City, Utah.

† Received by the Secretary, October 20, 1925.

With respect to the presence of sodium carbonate the effect of temperature is of possible concern. At 0° cent., sodium carbonate is soluble at the rate of 7.1 parts per 100 parts of water, while at 35° cent., the solubility is 51.0 parts per 100 parts of water. The warmer the canal water, therefore, the greater the extraction of the deflocculating salt.

Small irregularities in grade are of moment as is evidenced by the self-cleaning of clogged tile lines that have a grade too slight to provide an eroding velocity. The cleaning results from local erosion at the toe of the clogged part, due to the higher velocity of the water flowing down the short, steep section. Once eroded, the material is kept moving by virtue of the velocity of normal flow.

FLOOD FLOW CHARACTERISTICS

Discussion*

By Messrs. Merrill M. Bernard and C. S. Jarvis.†

Merrill M. Bernard,‡ Assoc. M. Am. Soc. C. E. (by letter).§—A review of the many contributions to engineering literature on flood flow and related subjects, impresses one with the fact that eventually there will develop, out of the maze of meager data and unsupported theory, a definite and clean-cut working knowledge of the relation between rainfall and the resulting run-off.

The engineer, having to do with drainage works, flood control, and the design of hydraulic structures and bridges, grasps eagerly at current articles pertaining to the subject, only to find, in many cases, that they fail to clear up many of the doubtful points.

The author has made available, in most satisfactory form, an exhaustive list of flood discharges, and has made some valuable suggestions. He presents a basic run-off formula, which should be followed up and utilized by those advocating this means of estimating flood flow.

In Table 9 are listed eighteen of the more important factors entering into an analysis of flood flow on any particular water-shed, many of them influencing run-off as much as the catchment area. Six of these eighteen factors

TABLE 9.

Constant factors.	Variable factors.
(1) Area of shed.	(1) Degree of efficiency desired.
(2) Shape of shed.	(2) Intensity of rainfall producing maximum flood to be considered.
(3) Arrangement of lateral drains.	(3) Frequency of rainfall producing maximum flood to be considered.
(4) Average slope of shed.	(4) Time of concentration.
(5) Average slope of drains.	(5) Effect of vegetable cover.
(6) Geographical location.	(6) Degree of improvement of lateral drains.
	(7) Losses: Evaporation, transpiration, and seepage.
	(8) Relation of storm center to water-shed.
	(9) Travel direction of storms.
	(10) Season in which storm occurs.
	(11) Relation of storm to storm period.
	(12) Available storage.

are constant and can be given definite values. The remaining twelve are variable, but in the study of a particular water-shed can be assigned maximum or average maximum values tending to give results that are on the side of safety.

* Discussion of the paper by C. S. Jarvis, M. Am. Soc. C. E., continued from September, 1925, *Proceedings*.

† Author's clo.

‡ Civ. Engr., Crowley, La.

§ Received by the Secretary, October 5, 1925.

The rational expression of the relation of rainfall to run-off, $q = C\,i$, in which, q is the run-off, in cubic feet per second, from a unit of catchment area, resulting from a rainfall of intensity, i, and C is a coefficient reflecting the physical characteristics of the water-shed, is becoming popular in the field of storm sewer design. It is based on a theory that maximum flow will result from a rain the duration of which is equal to the time of concentration.

The diagram, Fig. 17, illustrates the relation of the listed factors to the formula. In application, the value of C will range from 0.11 to 0.15, where the acre is the unit of area. The time of flow can be analyzed from head-water to outlet, which, in a co-ordinated system of drains, will take care of peculiarities in the shape of the area, the arrangement of lateral drains, etc. The writer recalls a water-shed made up in part of cut-over pine hills, having an average slope of about 12 ft. per mile, changing abruptly at the stream itself to a dense palmetto swamp, with a slope of about 0.4 ft. per mile. The outfall stream ran for 30 miles without adding 1% of area to the water-shed, due to the fact that the adjacent lands sloped away from the stream. In this case, it would be manifestly impossible to arrive at an intelligent result by considering the flood discharge as a function of the area alone.

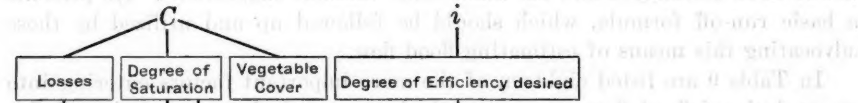

FIG. 17.—RELATION OF RUN-OFF FACTORS.

Assumptions must be made. If assumed values are applied to all factors, errors of judgment become compensative, and the designer has the satisfaction of knowing that he has dealt with all phases of his problem, even if he is in error as to some of them.

Constant Factors

Area of Water-Shed.—The area can be actually measured or estimated.

Shape of Water-Shed.—The shape of the water-shed is reflected in the analysis of the time of flow. A long narrow water-shed will produce a shorter concentration time than a broad, fan-shaped one.

Arrangement of Lateral Drains.—The topography usually decides this arrangement, but should future improvement demand a re-arrangement of lateral drains the fact should be considered.

Average Slope of Water-Shed.—This factor is subject to measurement and is necessary in determining the time of flow over the ground surface. It also aids in selecting what must be considered as the most remote part of the water-shed.

Average Slope of Drains.—This factor, too, is measurable, and is necessary in arriving at the time in transit through the drains.

Geographical Location.—The geographical location will place the water-shed under consideration in the proper weather characteristic group, and will permit the selection of a satisfactory rainfall intensity curve.

Degree of Efficiency Desired.—An economic study of the problem will determine a value for this factor. The flood selected as the maximum to be provided for carries with it the selection of a rainfall intensity curve which will be reached or exceeded with a given estimated frequency.

Intensity of Rainfall Producing Maximum Flood to Be Provided For.—Having determined the time of concentration at any point under consideration, the intensity of rainfall producing the maximum flood can be taken directly from the rainfall intensity curve.

Frequency of Rainfall Producing Maximum Flood to Be Provided For.—Where no other data are available, such works as "Elements of Hydrology", by Meyer, will provide a source from which can be selected an intensity curve of known frequency for a given geographical location.

Time of Concentration.—Although considerable error may enter into the estimate of the time of flow over the surface, it can be rather closely estimated after the run-off has reached the lateral and main drains. Where a knowledge of the flood flow is necessary in sizing the drains themselves, the increased flow, due to the decreased time of concentration, becomes apparent.

Effect of Vegetable Cover.—An inspection and study of the water-shed will determine that part which will be improved and cleared as a result of drainage or flood protection and that part which will remain for an indefinite period as woodland. Lateral water-sheds subject to such change should be dealt with, keeping the improved condition in mind.

Degree of Improvement.—Lateral Drains.—Where lateral drains are subject, within reasonable time, to extension or improvement, they should be so considered in estimating the time of flow through them.

Losses.—Evaporation, Transpiration, and Seepage.—Evaporation, transpiration, and seepage are considered in giving a value to C, vegetable cover, type of soil, and ground slopes being the controlling factors.

Relation of Storm Center to Water-Shed.—The available rainfall record is the basis for the selection of the rainfall intensity curve. The record of a heavy downpour may or may not represent the maximum intensity of the storm, depending on the relative location of the gauging station to the storm center. The record, therefore, must be assumed to represent the average intensity to be expected. This factor, like rainfall intensity, is governed by the law of probability, and can be grouped with it.

Direction of Travel of Storm.—Where the design of structures such as dams, spillways, and bridges, is involved, the path of travel of the maximum storm should be assumed to take the direction of flow, unless long-time records disclose the fact that such a condition is not probable.

Season of Year in Which Storm Occurs.—In localities subject to freezing weather and snowfall, the combination of conditions resulting in maximum run-off should be considered.

Relation of Storm to Storm Period.—The maximum storm should be assumed to occur during or at the end of the storm period when saturation is complete.

Available Storage.—What may be a storage basin to-day may be reclaimed for other purposes on the completion of a contemplated improvement. No storage should be depended on, which will not be available for a reasonable period.

The writer feels justified, therefore, in attempting to analyze and give values to the many factors involved in the problem of run-off and flood flow. Surely the results will justify the added effort.

C. S. JARVIS,* M. AM. SOC. C. E. (by letter).†—The generous response in discussions from members of the profession reflects the widespread interest in floods and their control. As aptly stated in a recent editorial:‡

"In many respects the serious study of the subject may be taken as dating from the memorable Central States floods of March-April, 1913. Long before that time many individual flood problems had been investigated, and in addition engineers specially concerned with stream problems had recognized and given attention to the more general questions of floods. Nevertheless, the common tendency was to look upon floods of disastrous size as catastrophic happenings, like earthquakes or lightning strokes, and the attitude of engineers was greatly influenced thereby. Each community not yet visited by a large flood was inclined to believe itself immune. Only after 1913 did it come to be generally understood that flood danger exists along every watercourse, and that there is enough system in the attendant circumstances to warrant abandoning the fatalistic attitude of waiting for a flood to come, and substituting therefor scientific study."

Such a view is quite opposed to the theory that "every river is a law unto itself, and unrelated to other streams even within the same area."

Just as C. E. Grunsky, Past-President, Am. Soc. C. E., has defined§ a simple relation between inches of rainfall and the percentage that will appear as

* Care, U. S. Bureau of Public Roads, Washington, D. C.

† Received by the Secretary, October 17, 1925.

‡ *Engineering News-Record,* June 4, 1925, p. 919.

§ *Transactions,* Am. Soc. C. E., Vol. LXI (1908), p. 514, and Vol. LXXIX (1915), p. 1165.

run-off, it seems reasonable similarly to seek the relation between drainage areas and their discharges under assumed or known rainfall intensities and surface conditions within the water-sheds. Variable coefficients have long been used to account for those functions, as illustrated by Talbot's formula; and in Kuichling's and Fuller's formulas the time element is included.

Run-Off Expressed as Percentage on the Myers Scale.—From the fundamental equation* of the paper:

$$Q = a\,V = 10\,000 \sqrt{M}$$

or,

$$q\,M = 10\,000\,p \sqrt{M}$$

Dividing by $100 \sqrt{M}$ there results,

$$100\,p = \frac{q}{100} \sqrt{M}$$

which gives the numerical value of the percentage; that is, the product of the run-off in second-feet per square mile times the square root of the drainage area in square miles, divided by 100, is the desired number. A single setting of the slide-rule affords a solution.

The ease with which such ratings may be ascertained is one point in their favor. For example, the Miami River, at Miami, Ohio, during the flood of March, 1913, had $q = 98$, and $M = 3\,957$. Then,

$$100\,p = \frac{q}{100} \sqrt{3\,957} = 61.7\%$$

as the rating for the discharge of that stream in terms of the modified Myers formula (the "fundamental equation" just noted).

At one stage of the writer's investigations the desired equation appeared to have the form, $x\,y = C\,k$, in which, C is a variable coefficient and k is a constant; but this equation was found to have only limited or local application. In small drainage areas of the Western States, affected by intense rainfall, it seems to have been demonstrated that up to certain limits the maximum flood is a constant irrespective of area. This may be explained by considering the usual storm, with its vortex traversing a path only a few miles in width at most. If from 30 sq. miles the run-off amounts to a depth of nearly 1 in. per hour, the maximum flood wave will register 18 000 sec-ft. Investigators in Wyoming have concluded that such a maximum seems to prevail for areas from 1 000 to 4 000 sq. miles. Reduced to the Myers scale these floods would rate 5.7% and 2.8% at the lower and upper limits, respectively. These values occur quite frequently throughout the Rocky Mountain District for both large and small areas.

The same tendency toward a constant maximum for varying areas may be observed by reference to Items Nos. 41 and 58 in Table 2.† The flood in question resulted from a precipitation estimated at 2 in. in 3 hours, over an area of 15 sq. miles which included only minor parts of the Honey Creek water-shed, yet yielded a run-off at a greater rate than that which obtained during March, 1913. The maximum intensity when reduced to the Myers scale is seen

* *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1553.

† *Loc. cit.*, pp. 1563–1564.

to be 57.2%, as compared with 61.7% at Miami during 1913. If properly interpreted, the observations during 1918 might be an index of possible floods in this region. The extensive investigations carried on by the Miami Conservancy District resulted in a design based on a run-off 40% greater than that recorded for 1913. This would rate at various critical points in the valley from 50 to 86% on the Myers scale.

As a practical application, the same intensity should be expected for the White River floods at Indianapolis, Ind., with such modifications as the dissimilar topographic, surface texture, and climatological factors may warrant. After diligent research the designers of the protective works at Indianapolis adopted as a basis a flood stage about 4 ft. higher than that of 1913. The Pueblo, Colo., flood of June, 1921, rated 15.2% for the total drainage area, or 28.5% for the part affected by the intense storm; the basis of design for the new channel is 60% in excess of these values, even though they might be regarded as indicating a 100-year, or even a 1 000-year, flood on well-known formulas.

Design of Drainage Structures for Highways.—Due to the ability of highway structures to withstand occasional overflows for brief periods without serious damage, more latitude is usually accorded in designing them than in railroad or municipal practice; thus, the 10-year maximum is often prescribed as the basis of reckoning in highway work. The trend seems to be toward using the rare flood as the datum, not necessarily in providing full capacity under the main structure, but rather using by-pass spillways corresponding with flood channels. This method precludes the possibility of a high embankment failing suddenly and without warning, as at Monterey, Mexico (see Item No. 493, Table 2*). In that instance, it was reported that the loss of human life exceeded 5 000, or double the casualties in the Johnstown flood. The sudden failure of the high embankment forming the bridge approach and impounding dam literally engulfed the peon districts of the city. Rare flood heights should be traced out so that they would be known well in advance and warnings issued accordingly.

The present situation in practice is well illustrated by the highway bridge specifications of the State of Ohio, approved June 10, 1925, in which Talbot's formula is used for small areas, and Fanning's and Kuichling's for larger drainage basins. These formulas are shown in Fig. 18 by the broken curves. The modified Myers curves of 5, 10, and 50% are also platted, together with the observed maximum flood data from Table 2 and Item No. 107† for Ohio; also representative maxima from California, Colorado, New York, and North Carolina. This demonstrates that the modified Myers formula may be substituted for a group of unrelated formulas, or may serve as a common denominator and a scale of comparison for them. Also, it applies equally well to any district if the coefficient is properly chosen.

The North Carolina State Highway Department has used the same method for Talbot's formula,‡ with C ranging from 0.2 on the coastal plains to 1.0 in

* *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1571.
† *Loc. cit.*, p. 1580.
‡ *American Highways*, July, 1925, p. 8.

the mountains. Reduced to the Myers scale, these ratings would be from 10% to 35%, as shown on Fig. 19.

Need for a General Formula.—The need for a general formula was evidently recognized by the American Railway Engineering Association, for its Special Committee seeking such an expression made progress reports in both 1909 and 1911.* Its final conclusion was that no existing formula could be

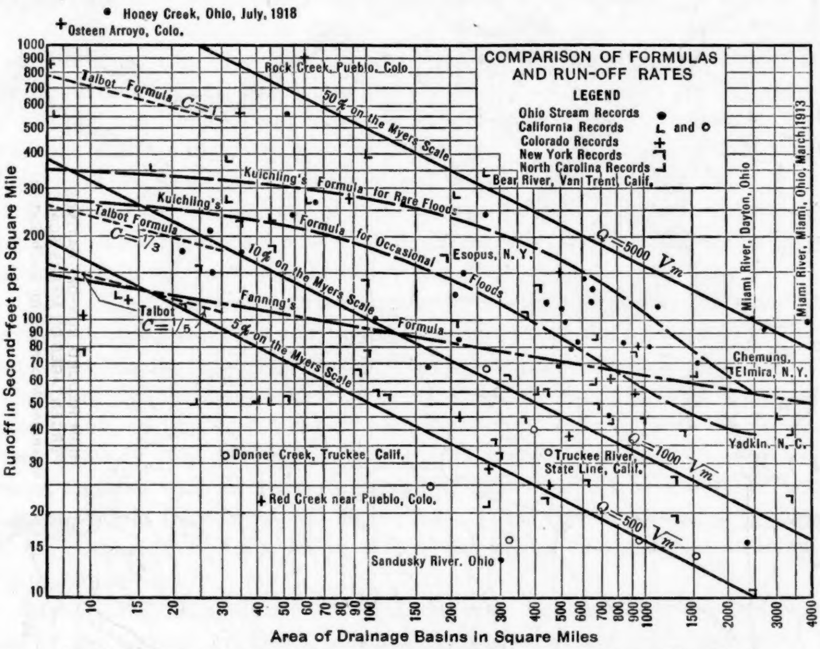

FIG. 18.—RELATION BETWEEN RUN-OFF RECORDS AND VARIOUS FORMULAS.

recommended for general application, but that Dun's tables best represented the relation between drainage area and run-off. It is significant that the average values of these tables rate 20% on the Myers scale. Specifically, they are 12% at 1 sq. mile, 21% at 7 sq. miles, and 19% at the upper limit, 6 500 sq. miles.

Applying a General Formula.—As Chief Engineer of the Santa Fé Railroad System, the late James Dun, M. Am. Soc. C. E., found it necessary to increase or reduce his tabular values before applying them to certain districts through which he operated, so that they rate for practical purposes from 5 to 50% on the Myers scale.

Other curves in Fig. 6† which show close parallelism in their upper ranges are those of Kuichling and Murphy, while that of Metcalf and Eddy will rate 10% at 10 sq. miles and 13% at 1 000 sq. miles, and gradually increase to 14% at 10 000 sq. miles. In effect, therefore, all these formulas involve the square root of the drainage area as an important factor, although this function may not be explicitly stated. Those with greater or less slope may be excel-

* "Bridge Engineering", by J. A. L. Waddell, Vol. II, p. 1111.

† *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1555.

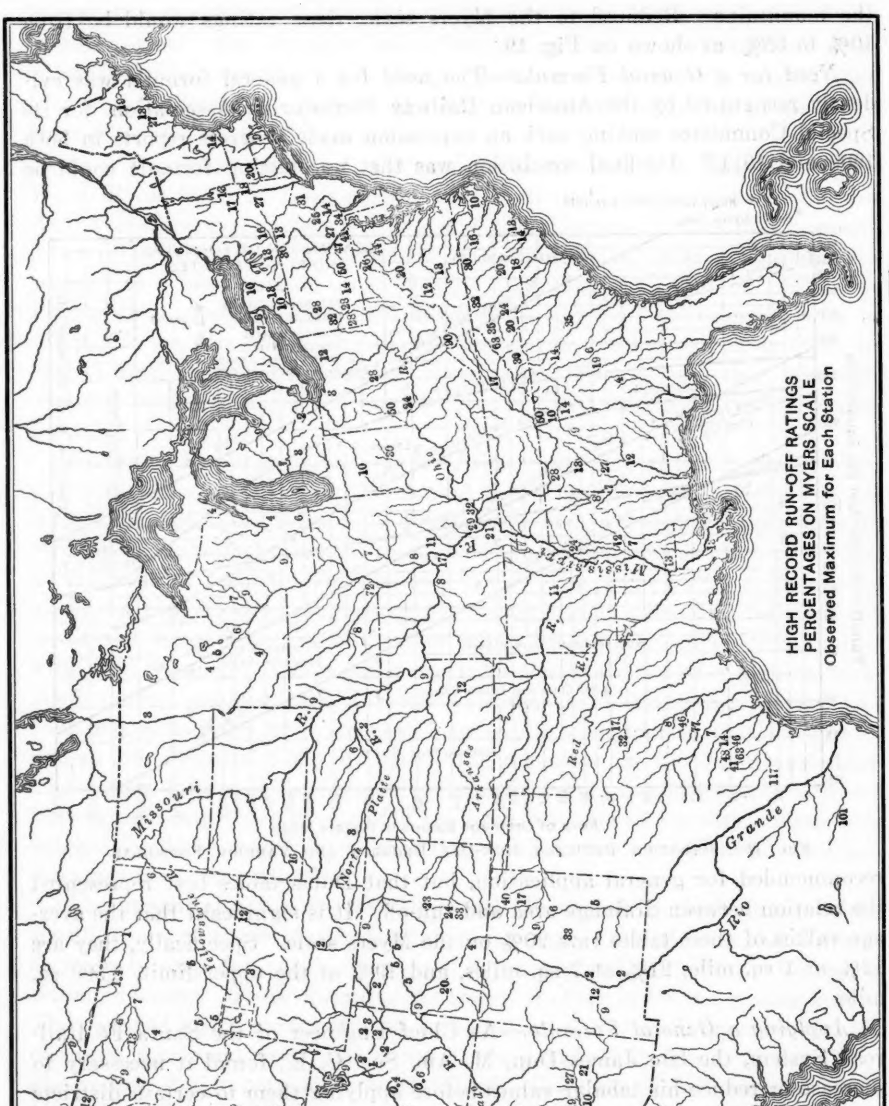

FIG. 19.—MAXIMUM OBSERVED RUN-OFFS (Q), IN TERMS OF PERCENTAGES OF MYERS' FORMULA $(10\,000 \sqrt{M})$.

lent for a limited range, but have to be discarded when they digress from the trend of the populated area as shown on Plate VIII,* and others must be utilized to cover the remaining territory, as shown in Fig. 18.

Simple tests are afforded by the river diagrams in Figs. 20 and 21. If the slope of the Myers curve is correct, it should parallel the graphs of typical streams. Then the determination of the rating at any section would provide an index for all other points along its course. Thus, the Monongahela and the Ohio are fairly consistent in ratings from 25 to 35%, likewise the Sacramento and Columbia (Fig. 21). The abrupt changes of slope for the Sweetwater and the Pequannock (Fig. 20) herald the influence of the humid coastal regions in the one case, and of channel storage and flood-plains in the other. The Willamette River increases very definitely as it invades the moist valley near Portland, Ore., and the Truckee has paid toll to the desert (Fig. 21).

Discussions.—Mr. Hartwell† deplores the apparent neglect of the time element. This factor is involved with others, additive or subtractive, in the value of the coefficient to be adopted in any given instance.

Mr. LaRue‡ rendered valuable service both before and after the paper was written, drawing from his wide experience, subjecting the formula to practical tests, and lending untold encouragement to continue the tedious quest. He points out that the maximum flood of the Colorado River at the Laguna Dam is only 80% of that registered at Lees Ferry from only 58% of the drainage area. This is only natural, for the greater channel storage and percolation losses, together with the reduced slopes and desert conditions below the Grand Canyon, would smooth out the flood peak. These factors warrant the change in the coefficient, C, from 760 to 460, or the rating on the Myers scale, from 7.6 to 4.6 per cent.

Mr. Grover§ has pointed out the limitations that must be observed in the use of any general or special formula dealing with natural phenomena. In the course of his official duties he secured through the District offices a careful checking of all the flood data in the paper derived from the U. S. Geological Survey, and completed all records to date. Under his guidance the Society's Special Committee on Flood-Protection Data should evolve some systematized data from the chaotic mass that now exists. The writer's efforts have been co-ordinated wherever practicable with the plans of this Committee; he realizes that the task of digesting the available records must be accomplished, if at all, by some such agency.

Mr. Okazaki‖ calls attention to the inevitable results of diking and channel rectification without due regard to by-pass functions and lateral storage. There are meadows and other lands that would derive positive benefit from inundation by a turbid crest, yet they are enclosed by great levees which insure the maximum severity of flood stage for the lower valleys.

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* *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1555.

† *Loc. cit.*, April, 1925, Papers and Discussions, p. 678.

‡ *Loc. cit.*, p. 679.

§ *Loc. cit.*, p. 680.

‖ *Loc. cit.*, p. 682.

FIG. 19.—MAXIMUM OBSERVED RUN-OFFS (Q), IN TERMS OF PERCENTAGES OF MYERS' FORMULA $(10\,000\sqrt{M})$.

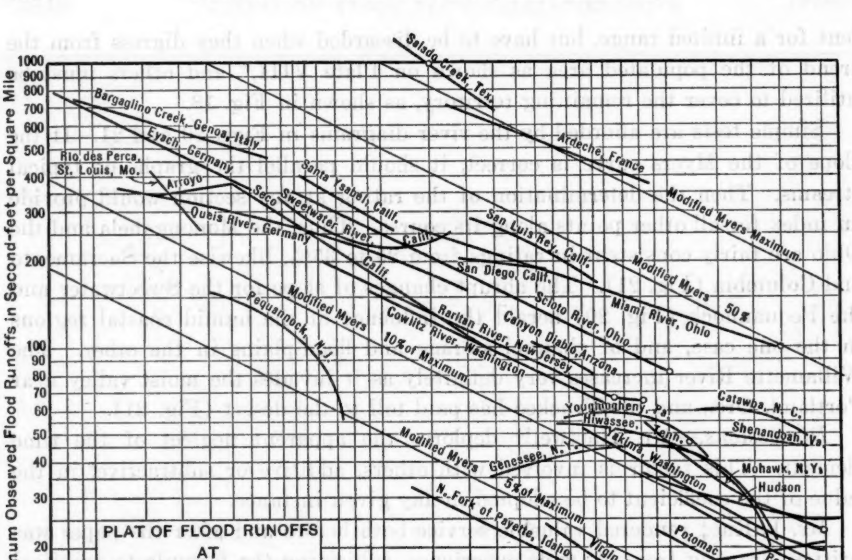

FIG. 20.—COMPARISON OF RIVER DISCHARGES.

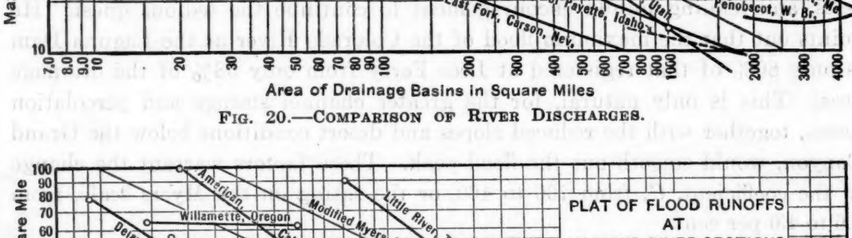

FIG. 21.—COMPARISON OF RIVER DISCHARGES.

Mr. Sherman* gives valuable suggestions regarding the elements of flood frequency and rainfall intensity. The highway bridge specifications of the State of Ohio, approved June 10, 1925, have included such basic data as the following:

"Thirty-three weather bureau stations in Ohio selected at random, covering a period of observation averaging 24 years, show that a rainfall exceeding 2½ in. in 24 hours and 3 in. in 48 hours occurs more frequently than once in 5 years, also, that a 3½-in. rainfall in 24 hours and a 4-in. rainfall in 48 hours occurs more frequently than once in 20 years.

"Also six stations at which hourly rainfalls have been observed for an average period of 40 years show that a 1-in. rainfall occurs more frequently than once every two years, and a 2-in. rainfall (per hour) more frequently than once each 16 years."

The importance of such data as a basis for analysis and scientific design is being recognized as never before.

Mr. Winsor† has doubts as to the applicability of any formula to the mud slides and rolling débris which occasionally precede a flashy flood. His careful study of this subject and his direct observations during the storm of August, 1923, along the Wasatch Range, Utah, lend additional weight to his opinions, yet, somehow, a bridge engineer must design a structure that may span such a grinding torrent. The photographs of flood effects from that storm (Figs. 3, 4, and 5‡), were kindly contributed by Mr. Winsor for publication with the paper.

As illustrating how a formula may apply to certain phases of desert run-off as well as to the wooded areas, consider the floods in the Rio Grande at El Paso, Tex., during the early part of August, 1925. On July 30 and 31, the U. S. Weather Bureau observed the basic data on which Fig. 22 is based. Precipitation as great as 6.3 in. occurred over a great area, centering at Hermosa, N. Mex., 30 miles west of the Elephant Butte Dam, and averaging a depth of 3 in. over 1 000 sq. miles, or more. Part of the drainage was toward desert sinks, and nearly one-half of that in the Rio Grande Valley discharged above the dam; yet about 10 800 acres were flooded when the smoothed-out crest reached El Paso, with an estimated damage of $100 000, despite all efforts at control.

Table 10 is based on data§ supplied by L. M. Lawson, M. Am. Soc. C. E., Project Superintendent, U. S. Reclamation Service, El Paso, and shows not only the desilting and flood protection produced by ample storage, but also the sixfold increase of minimum dependable yearly flow. The last three columns have been added by the writer.

The dangers outlined in the paper under the sub-title, "The Wooded Channel",‖ were exemplified in the havoc wrought near El Paso from a flood crest of 8 400 sec-ft., or nearly one-half the previous record.

Is it merely a coincidence that the maximum ratings on the Myers scale for San Marcial and El Paso are so nearly comparable, when the active drain-

* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 902.

† *Loc. cit.*, p. 906.

‡ *Loc. cit.*, December, 1924, pp. 1549 and 1551.

§ *Engineering News-Record*, Vol. 95, September 3, 1925, p. 372.

‖ *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1558.

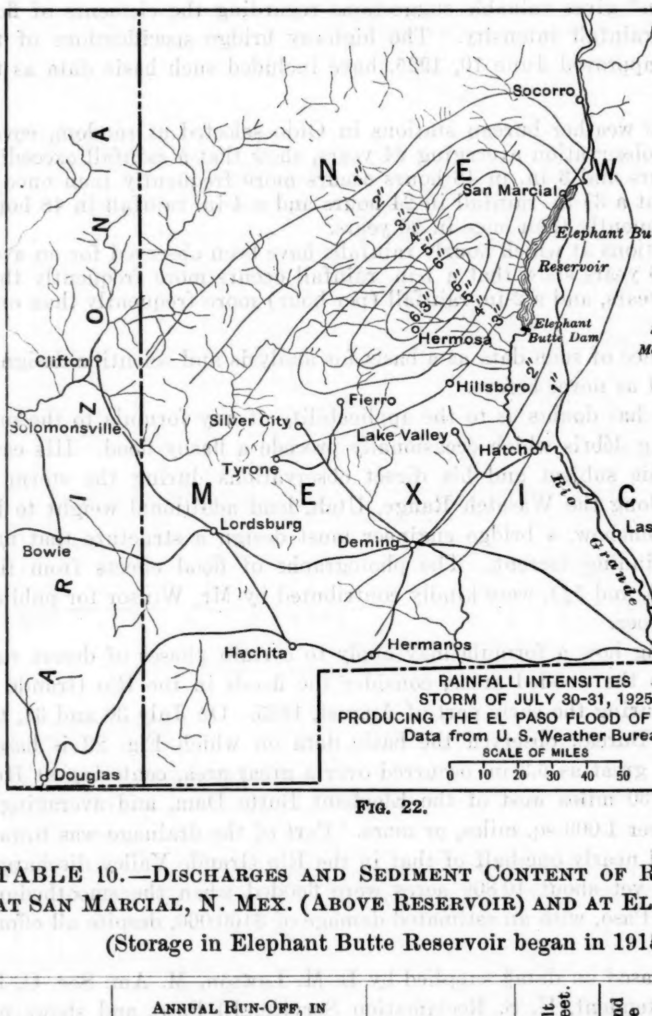

FIG. 22.

TABLE 10.—DISCHARGES AND SEDIMENT CONTENT OF RIO GRANDE AT SAN MARCIAL, N. MEX. (ABOVE RESERVOIR) AND AT EL PASO, TEX.

(Storage in Elephant Butte Reservoir began in 1915.)

Place and period.	Annual Run-Off, in Acre-Feet. Maximum.	Minimum.	Mean annual.	Maximum discharge, in second-feet.	Mean annual silt content, in acre-feet.	Run-off, in second feet per square mile (q)	Drainage area, in square miles (M).	Maximum rating on the Myers scale, percentage.
San Marcial. 1895-1924........	2 422 008	201 000	1 225 727	33 000 (Oct. 11, 1904)	19 000	1.10	30 000	1.90
El Paso, 1889-1914.	2 010 000	50 700	925 400	17 000 (Oct. 15, 1904)	15 300 (1906-09)	0.45	38 000	0.89
El Paso, 1916-1925.	870 418	348 951	685 232	8 400 (Aug. 2, 1925)	480	1.05	8 000 (Below reservoir)	0.94

age areas are considered? If this method of comparison means anything, the plane of a 20 000 sec-ft. stage should be traced out below El Paso, so that the limits of the danger zone may be known well in advance in case the recent storm intensity should be repeated, with the center at Hillsboro, N. Mex., instead of Hermosa, or else a storage reservoir should be established for the joint purpose of flood control and power development below Elephant Butte.

On July 24, 1924, the intensity of a local storm near Hondo, N. Mex., about 100 miles east of Elephant Butte, equalled or exceeded the one recorded in Fig. 22. The writer had occasion to observe the effect on a newly constructed Federal Aid road project, and reported in part as follows:

"The drainage structures on 70% of the 16-mile length were overtaxed by 10%, and on the remaining mileage they were not required to perform full duty. As the crest which caused overtopping of the road grade evidently lasted only a few moments, the damage was insignificant."

The culverts were proportioned by tables comparable with Talbot's $C = 0.2$, or from 2 to 6% on the Myers scale for the areas considered, with due regard to special local conditions. Spillways or dips in the grade proved very effective in safeguarding the major structures. At one of these locations traffic was halted for a few hours; but even desert habits of run-off have proved amenable to rational design.

Mr. Hinckley* and the writer have evidently traversed parallel paths of investigation and their conclusions are in close agreement. The fundamental equation of the paper may be written as:

$$Q = \frac{10\,000\,M}{\sqrt{M}}.$$

This shows that the run-off from each square mile which rates 100% on the Myers scale is 10 000 cu. ft. per sec. for 1 sq. mile; 3 162 cu. ft. per sec. for 10 sq. miles; 1 000 cu. ft. per sec. for 100 sq. miles; and 316.2 cu. ft. per sec. for 1 000 sq. miles of drainage area.

The Musi River, India, is reported to have discharged at the rate of 493 sec-ft. from each of the 862 sq. miles above Hyderabad.† The corresponding position on the Myers scale is 144 per cent. The four items thus far discovered that exceed the 100% limit adopted for convenience of reckoning ought not to detract from its applicability.

The discharge of 10 000 sec-ft. from 1 sq. mile is equivalent to 1 in. of rainfall run-off every 4 min., and requires special extreme conditions or coincidences, such as a deluge on melting snow, or exact synchronism of violent flood waves.

By far the majority of ratings fall below 50%; the 10% line is the axis of gravity, apparently, on Plate VIII; and many streams stay consistently below 5% throughout their courses, as illustrated on Figs. 20 and 21.

Mr. Lane‡ has furnished an instructive elementary analysis of many factors which the writer recognized and enumerated, but could not develop without obscuring the central idea. The results derived from due consideration

* *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1118.

† Technical Repts., Miami Conservancy Dist., Pt. IV, p. 66.

‡ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1121.

of the time, climatic, and physical elements will surely be expressed numerically; and if so, they will take a definite rating on the Myers scale. As an example, the rare floods at Pueblo, Colo., may rate 15% or even 25%, according to recent design data for protective works built under Mr. Lane's direction; but the frequent floods will probably rate less than 5 per cent. Between these values lies a wide range for refinements such as analytic or rational methods will produce. The Myers scale affords a direct method of first and, perhaps, second approximations, but does not take the place of the later extended research and the exercise of mature judgment. It is essentially a common denominator for diverse expressions.

Mr. Matthes* has made an attractive presentation of governing principles for this subject and has furnished one of the best practical tests that could be devised for the Myers scale. The Susquehanna, Ohio, and Tennessee Rivers are shown in Table 9† to have ratings of 45.2, 32.1, and 27%, respectively, for comparable drainage areas with a century of record or observation on which to depend. It would be interesting to extend the comparison to some of the typical tributaries of these and other rivers.

The scope of the work and the arrangement of the data outlined by Mr. Matthes, as Secretary of the Society's Special Committee on Flood-Protection Data, lend assurance that its efforts are well directed. Although this promises to be one of the most fruitful fields for engineering endeavor the activities have been halted by lack of funds. From the standpoint of insurance the expenditure of one-millionth of the values at stake would be repaid many-fold during the next year of violent storms.

Mr. Kurtz‡ has displayed the same conception of the best form for presenting the data as first appealed to the writer. This had to be deferred because of time limitations, and because the Society and the Government possess the proper agencies to evolve the elaborate analyses and comparisons. From a tangled mass of unrelated data the transformation must proceed step by step. The imperfect, incomplete, somewhat fragmentary assemblage which Table 2 represents may be most useful in showing that most of the work still remains to be done. The methods of interpolation and exterpolation used by Mr. Kurtz have proved basically sound in many cases observed by the writer.

Mr. Comstock§ opposes the practice, almost universally adopted, of relating the rate of run-off to the area of the drainage basin. Some standard or well-known formulas use few, and some involve many, factors of influence; but where is there an expression that omits consideration of the drainage area? This factor may be measured, and it remains nearly constant; but most of the others depend on probabilities and estimates at best.

Mr. Merriman‖ develops the idea that Table 2 is not so much a record of floods as of maximum observed discharge. This thought was clearly impressed on the writer during the tedious process of tabulation, for many streams would rate on existing records far below the probable high flood intensity, and por-

* *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1133.
† *Loc. cit.*, p. 1135.
‡ *Loc. cit.*, p. 1139.
§ *Loc. cit.*, p. 1142.
‖ *Loc. cit.*, p. 1144.

trayal on the Myers scale gave a basis of comparison not otherwise furnished. The river diagrams shown in Figs. 20 and 21 result from merely connecting the points representing run-off intensities on the Myers scale for various sections. The Mohawk River is there shown to yield only about one-third of the Miami discharge from an equal area.

The Pueblo flood of 1921 taught engineers more about Arkansas River run-off capacities than did the data from previous decades, and multiplied the run-off given by previous records by the factor, 9. Likewise, the maximum observed on the Santa Ana River, in California (Item No. 299, Table 2*) has increased more than fivefold during the past ten years and the San Gabriel (Item No. 331, Table 2*) has quadrupled its rating. Others have doubled since the paper was first written. On the other hand, the great mass of records, probably 90%, has undergone only minor changes, if any, in twenty years.

If Esopus Creek might be used as an index to the Catskill water-shed the record run-off of September, 1924, ought not to be regarded as a radical upsetting of data. It rated only 23.9% on the Myers scale (Item No. 341, Table 2*), as compared with 27.0% for the same stream at Saugerties, N. Y. (Item No. 437, Table 2†). The Chemung, at Elmira, N. Y., reached 30.4% in 1889 (Item No. 700, Table 2‡), and the Mohawk rated 16.8% at Cohoes, N. Y., while the Hudson was 17.9% at Mechanicville, N. Y., during 1913.

The Schoharie adjoining the Esopus water-shed reached only 18.9% at Prattsville, N. Y.; but with a similar storm centering differently there seems to be no insurance against its rating 25% or more. The same might apply to the Catskill Creek Basin or the Rondout as described and illustrated in the discussion by Mr. Honness.§

After scanning the percentages recorded on the map in Fig. 19, the basic data in Table 2, and the information on Fig. 18, a generalization might be ventured regarding the St. Lawrence drainage from Wisconsin to New York: It seems to range below 15% on the Myers scale, averaging nearly 6%, the same as Niagara River. This may be accepted as a first approximation, and further research may show that capacities of one-half or even one-tenth would suffice on several streams in this area. Furthermore, it may be deduced by the same process that the Mississippi drainage in the same States would rate double or, in some cases, five times those determined for the St. Lawrence water-shed. The disparity is evidently not in rainfall but due in part to the presence of lakes among glacial débris in the one area, and the increased slopes in the other.

The writer must differ with Mr. Merriman's views‖ regarding the reliability of flood flow data prior to 1900, believing that there is much to be gained from even the crude approximations which necessarily served on the frontier. No doubt, present methods will be regarded as inadequate by later investigators. However, there is thorough agreement as to the value of Nature's historical monuments along each watercourse. The rings denoting annual growth

* *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1569.

† *Loc. cit.*, p. 1570.

‡ *Loc. cit.*, p. 1574.

§ *Loc. cit.*, May, 1925, Papers and Discussions, p. 904.

‖ *Loc. cit.*, August, 1925, Papers and Discussions, p. 1144.

of timber may by their varying widths indicate the trend of climatic changes and the extremely dry or humid years of past ages. The record afforded by the gravel bars, detrital cones, and laminated sediment should be interpreted by expert geologists. Otherwise, many signs may be misinterpreted; for example, the back-water effects from a temporary impounding dam at a junction, or a landslide such as occurred on the Gros Ventre River, Wyoming,* June 23, 1925, or the successive stages occupied during the process of erosion.

Mr. Fuller rendered his first service to this paper when he published his notable work on "Flood Flows"† in 1914. Mr. Hazen's discussion of that paper included a United States map‡ with Fuller's coefficients recorded on various streams; and when the writer observed that Central Texas was rated at less than one-tenth of what prevailed in the Pacific Northwest, he began his definite quest for a basic formula capable of more satisfactory comparison. The paper is without doubt a composite of the experience, observations, and analyses of many in the field of research; and among the prominent influences are the works of Messrs. Fuller, Hazen, Metcalf and Eddy, and the late Emil Kuichling, M. Am. Soc. C. E. Acknowledgments have been made wherever practicable, but there are numerous factors which cannot be defined.

The only reply that will be ventured to the objections outlined by Mr. Fuller§ regarding the slope of the Myers formula is the suggestion that his own curves be platted on Plate VIII for any small-area floods worthy of the name, and that he observe how far above the large-area floods these lines will fall.

Mr. Parsons‖ evidently has understood the writer's purpose more clearly than most readers, for his suggestion as to an additional factor, K, to represent the influence of physical and other characteristics of drainage areas coincides with the original conception of a percentage scale on which all streams may be rated. It would give a product, $CK = C'$; or in other words, a coefficient in which are combined and reflected all the factors and influences that claim consideration.

The discussions may be the source of the greatest benefit derived from this paper. The writer gratefully acknowledges the contributions of Messrs. Follansbee, Honness, Cross, Horton, and Bernard. Although these discussions do not call for special comment, the data contained therein have added greatly to the value of the paper. Thanks are also due to the many engineers—in the U. S. Geological Survey and elsewhere—who have so kindly checked and corrected the statistics regarding flood flow.

The task of segregating and digesting the available data should be carried on, not by individuals who will duplicate each other's efforts, but by a well-equipped organization such as the Society's Special Committee on Flood-Protection Data, working hand in hand with the U. S. Geological Survey.

* *Engineering News-Record*, Vol. 95, September 17, 1925, p. 467.

† *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 681.

‡ *Loc. cit.*, p. 629, Fig. 8.

§ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1145.

‖ *Loc. cit.*, p. 1153.

STREAM POLLUTION

A SYMPOSIUM

Discussion*

By Messrs. Richard H. Gould and J. Frederick Jackson.

Richard H. Gould,† M. Am. Soc. C. E. (by letter).‡—The methods outlined by Mr. Streeter§ will be of assistance to those working on stream pollution problems. The mathematical treatment is interesting and is evidence of much careful work. It must be confessed, however, that for one a little rusty in mathematics it is somewhat difficult to visualize fully all the aspects disclosed and to obtain a clear understanding of the relative importance of the factors involved. This does not detract, however, from the merit of the paper. The results secured are well worth the extra effort involved in following these new methods. The work on the Ohio and Illinois Rivers and the studies here reported are of great importance, and the men who are responsible for the results deserve great credit.

A number of points as to the practical application of the methods involved will bear comment. Some of these are brought out in the paper, but it may be well to emphasize them further. One of the assumptions in this method of computation is that a load of pollution entering a stream will exert a demand on the dissolved oxygen of that stream in accordance with certain definite laws of biochemical action. The demand exerted by this polluting material proceeds in an orderly manner, depending on the time elapsed, temperature, etc. The duration of contact with the waters to any down-stream point is presumed to be that of the time of flow of the water in the stream. This assumption is essentially correct where the polluting matter is free from settleable solids so that all material producing a demand on dissolved oxygen moves down stream with the water.

A serious error would appear to be introduced if parts of the polluting matter can separate out in the form of sludge on the bed of the stream. This latter condition more often than not seems to be the one actually met. Very few natural streams receiving polluting matter maintain velocities at all places and times that will prevent deposition of sewage solids. Depositions

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* This discussion (of the Symposium on Stream Pollution, presented at the meeting of the Sanitary Engineering Division, Cincinnati, Ohio, April 23, 1925, and published in November, 1925, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† With James H. Fuertes, New York, N. Y.

‡ Received by the Secretary, May 28, 1925.

§ *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1829.

occur most readily where the stream is sluggish or the discharge is into tidal waters with reversing currents and slack water. The problem is thus complicated in most cases by the condition that, whereas part of the polluting matter will remain in suspension or solution and, consequently, its total oxygen demand within a definite time of flow can be calculated with a satisfactory degree of accuracy, the other part will exert its demand, within a particular section, for an uncertain period of time.

Mr. Streeter has discussed the effect of sludge deposits and states that the methods proposed are not accurate where such deposits exist. As a practical consideration, it is probable that the effect of settleable solids is nearly always present to a greater or less extent when raw sewage is discharged into natural watercourses. Even where no deposition occurs, the slow rolling of solid particles along the bottom may thus lengthen their period of contact and consequent oxygen demand within a given stretch of river. The relative importance of these two classes of material producing oxygen demand will vary in any particular case. Not much is known about the demand exerted by sludge and sludge deposits, but enough to indicate that this demand may have a considerable effect.

Another factor which adds to the importance of separate consideration of settleable solids is that solids separating out in the stream during the winter may lie more or less dormant, in regard to exertion of an oxygen demand in the stream. With the coming of the warmer summer temperatures, the rate of decomposition will increase and the decomposition of accumulated deposits will add to the oxygen demand exerted by the current summer solids. The maximum rate will be exerted at the most critical time of the year as far as the condition of the stream is concerned. The extent of this effect will depend on conditions in the stream.

The relative importance of the separate consideration of settleable and non-settleable solids may be indicated very roughly by a hypothetical case as follows: If it is assumed, for example, that raw sewage is being discharged into a sluggish stream whose time of flow to a large volume of diluting water is two days, the oxygen demand of the sewage on a per capita basis can be calculated as follows:

If the 20-day oxygen demand of raw sewage is 0.24 lb. per capita, at a water temperature of 23° cent., about 42% of the 20-day demand, or 0.10 lb. of oxygen per capita per day, would be exerted within a 2-day period by the decomposing matter, assuming that both the settleable and non-settleable solids were affected at the same time. In the particular case under consideration, however, it is assumed that the sluggish stream will permit the deposition of sewage solids, and that these will remain on the stream bed for a long time. Based on the ordinary oxygen demand test, about one-third of the oxygen demand of raw sewage is removed by sedimentation. The demand of the liquid parts of the sewage within the section considered would be then 66% of the figure for raw sewage, or 0.066 lb. per capita per day. The 20-day demand of the current settleable solids, if all were deposited and remained on the stream bed, would be one-third of 0.24 lb., or 0.08 lb. per capita. If this were increased 50%, a purely arbitrary figure, owing to the

demand of old sludge banks in the stream becoming more active under the warmer summer temperatures, the total sludge demand would then be 0.12 lb. per capita, and would all be exerted in the section under consideration. The total of both the liquid and the sludge parts would be a demand of 0.186 lb. per capita per day, which is 86% above that derived on the assumption that no separation of solids took place. Absolute values will vary with existing conditions, but it would seem essential in most cases to give separate consideration to settleable and non-settleable matter.

A few engineers have felt for some time that the difference between the demand of raw and settled sewage may not be entirely brought out by the 5-day incubation test for oxygen demand. It has seemed that settled sewage would exert a higher percentage of its total demand in a period of 5 days than raw sewage with its coarser particles. How important this distinction is if there is a difference is simply a matter of speculation. In this connection, it is interesting to note in the paper by Mr. Theriault* that even after 20 days a very considerable oxygen demand may be exerted, that is, the 0.24 lb. per capita oxygen demand which has been mentioned, may not represent the total, and the solids deposited in the stream may exert more of a demand than is now estimated.

The influence of sludge deposits is indeed a disturbing factor and extends to the values derived for the aeration factor, K_2. This factor, it is understood, expresses the net aeration taking place in a stream, that is, it is the total aeration taking place through the water surface less the oxygen demand of the sludge deposits in the stream. This being so, coefficients derived from rivers of the same degree of turbulence, cross-sectional areas, depths of flow, velocities, etc., might be entirely different, depending on the relative amounts of oxygen demand exerted by the sludge deposits in the different streams. If the full significance of this coefficient is clear to the writer, it would thus appear that a coefficient derived from a sluggish stream receiving raw sewage pollution might not apply to this same stream when this sewage had been treated by sedimentation or otherwise.

Definite values for the effect of sludge deposits and of the amount of aeration are difficult to determine. There is one equation with two unknowns. If the rate of true aeration in a particular case was known, the demand of the sludge could be calculated, or *vice versa*. It would seem that it might be easier to derive values for re-aeration rather than for sludge deposits of uncertain age and extent. In most problems where complete data are lacking, aeration values would be the easier to apply. The three fundamental factors would be: The area of water surface, which is easily obtained; the percentage of saturation of the water, which may be measured without difficulty; and the degree of turbulence of the water. The degree of turbulence would be the most uncertain item, but it would seem that in the course of time a sufficient number of limiting values could be obtained to be of considerable aid to judgment in selecting the proper value.

The basing of aeration figures on the area of water surface rather than on the time of flow or other considerations involving the depth of water,

* *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1819.

velocity of flow, and stream characteristics, has the advantage that it may be applied both to flowing streams and to lakes and harbors where the effect of currents is not so noticeable. In most natural waters, particularly flowing streams and tidal estuaries, there seems to be little evidence that stratification of dissolved oxygen exists to any considerable extent. Where the methods outlined by Mr. Streeter are not applicable it would simplify computations if all considerations other than the area of water surface and percentage of saturation could be omitted. The particular reference in mind is that the consideration of depth of water as affecting the rate of diffusion of oxygen to the bottom layers does not seem essential. The area of water surface is readily obtainable, whereas mean depths and velocities are more difficult to secure. A convenient unit, such as pounds of oxygen absorbed per day per 1 000 sq. ft. of water surface, might be used.

An interesting approximation is suggested by the material presented by Mr. Streeter in connection with rates of re-aeration. It is brought out that the rate of solution of oxygen at the surface is directly proportional to the existing saturation deficit. It is also shown that the saturation deficit of water at 5° cent. is about 50% greater than the saturation deficit at 25° cent. when both liquids are completely devoid of dissolved oxygen. According to this law, it would be expected, neglecting the effect of temperature on the rate of solution, that oxygen would be absorbed by the colder water at a rate about 50% greater than by the warmer water. Mr. Streeter, however, gives curves, Fig. 4,* showing the effect of temperature on the rate of oxygen absorption. These corrections to a large measure counteract the effect of the greater saturation deficits that are possible in colder temperatures. The relative figures for three selected temperatures are given in Table 15.

TABLE 15.—VARIATION OF RATES OF AERATION WITH TEMPERATURE.

Temperature, in degrees centigrade.	Saturation deficit at 0%, in parts per million.	Ratio of aeration rate to that at 20° cent.	Column (2) times Column (3).	Figures in Column (4) expressed in percentage of value at 20° cent.
(1)	(2)	(3)	(4)	(5)
5	12.80	0.79	10.1	110
20	9.17	1.00	9.17	100
25	8.38	1.10	9.22	100.3

The figures in Table 15 indicate that the net variation in the rate of absorption of the waters from the highest ordinary summer temperatures to those closely approaching the lowest in winter is not much more than 10 per cent. The difference in the range in which engineers are mostly interested is considerably less and, in any event, within the limits within which the values of aeration and oxygen demand of the waters in a stream can at present be determined. With the present knowledge of the values involved, would it not be permissible to express values for re-aeration in terms of percentage of

* *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1832.

saturation and neglect temperature corrections, except as they are involved in determining the percentage of saturation of the liquid?

The intent of this discussion is not to detract from the importance of the work which is described by Mr. Streeter. This work marks a distinct advance in the science of the study of problems relating to the condition of polluted waters. The application of the methods outlined, however, must be made with caution in order that the fundamental assumptions in the theory developed may be in accord with the actual conditions in the particular problem to be solved.

J. FREDERICK JACKSON,* M. AM. SOC. C. E. (by letter).†—Following Mr. Hoskins' suggestion, a comparison of the results obtained by him with some of those obtained on the Naugatuck and Hockanum Rivers in Connecticut is herein attempted.

Taking the seasonal variation, the total count for bacteria in summer below Chicago, Ill., was 24 828 billions and for *B. coli,* 428 billions per capita per day; in winter, it was 4 257 billions and 42 billions, respectively. The results for Peoria, Ill., Cincinnati, Ohio, and Louisville, Ky., are similar, that is, the number of both total bacteria and *B. coli* was much larger in summer than in winter.

Below Waterbury, Conn., on the Naugatuck River, for summer the total bacteria were 4 839 billions and the *B. coli,* 1 068 millions per capita per day; and for winter the count was total bacteria, 429 billions, and *B. coli,* 1 334 millions. The results below Torrington and Ansonia, Conn., show the same relation in numbers of bacteria for summer and winter. While the results obtained on the Naugatuck River show higher numbers of bacteria in summer than in winter, the percentage increase is much less than on the Ohio and Illinois Rivers and the *B. coli* actually were greater in number in winter than in summer.

The differences are shown more clearly in the following maximum and minimum monthly counts of bacteria per cubic centimeter:

Below Torrington:	
In June: Maximum total bacteria	162 000
B. coli	37
In December: Maximum *B. coli*	1 000
Total bacteria	25 000
Below Waterbury:	
In June: Maximum total bacteria	347 000
B. coli	28
In December: Maximum *B. coli*	622
Total bacteria	23 000
Below Ansonia:	
In October: Maximum total bacteria	985 000
B. coli	8
In March: Maximum *B. coli*	280
Total bacteria	34 000

* Cons. Engr., New Haven, Conn.

† Received by the Secretary, June 20, 1925.

These results show a consistently higher number of *B. coli* for the winter than for the summer months. Mr. Hoskins states:*

"Such bacteria tend to increase in numbers in the receiving stream for a short period and then decrease at orderly rates as the time from the point of maximum density is increased. These rates of decrease * * * being most intensive during the warmer months and under conditions where the density of bacteria was greatest."

On the Naugatuck River, the results in bacteria per cubic centimeter were:

Above Torrington:

Total bacteria	6 000
B. coli	103

3 miles down stream below the city:

Total bacteria	123 000
B. coli	289

5.6 miles farther down stream:

Total bacteria	208 000
B. coli	289

Above Waterbury:

Total bacteria	30 000
B. coli	92

4.7 miles down stream below city:

Total bacteria	302 000
B. coli	164

2 miles farther down stream:

Total bacteria	424 000
B. coli	189

Above Ansonia:

Total bacteria	504 000
B. coli	28

5 miles down stream below city:

Total bacteria	479 000
B. coli	121

These results do not confirm those obtained on the Ohio and Illinois Rivers in relation to the time factor and the decrease in number of bacteria or as to the largest rate of decrease being during the warmer months.

Applying the formula† proposed by Mr. Hoskins for computing the number of *B. coli* per cubic centimeter contributed to a stream by sewered populations and using the mean annual flow of the Naugatuck River and the coefficients used by him, the results shown in Table 16 are obtained.

The difference between the computed and the actual numbers of bacteria is so great as to cast suspicion on the general applicability of the formula.

In order to determine whether industrial conditions on the Naugatuck River were responsible for the differences in the seasonal variation and the number of bacteria as computed and as actually found, the same computations

* *Proceedings*, Am. Soc. C. E., November, 1925, Papers and Discussions, p. 1855.

† *Loc. cit.*, p. 1846.

were made for Rockville, Manchester, and East Hartford, Conn., on the Hockanum River where the industrial wastes entering the river are not such as should affect materially the bacteria or plankton.

TABLE 16.—Comparison of Computed and Actual Number of Bacteria.

	Total Bacteria.		*B. coli.*	
	Computed.	Actual.	Computed.	Actual.
Torrington, Conn	1 175 000	123 000	28 088	289
Waterbury, Conn	1 302 179	802 000	25 590	164
Ansonia, Conn	167 721	479 000	8 295	121

Above Rockville the maximum total bacteria was, in August, 9 800, and the maximum *B. coli* was, in March, 10; 5.4 miles down stream below the city the maximum for both was, in March, total bacteria, 4 000 000 and *B. coli,* 100 000.

Above Manchester, the maximum for both was, in June, total bacteria, 765 000 and *B. coli,* 1 000; 4.6 miles below Manchester, the maximum for both was, in March, total bacteria, 620 000 and *B. coli,* 10 000; 3.5 miles down stream at the mouth of the river, the maximum total bacteria was, in March, 780 000 and for *B. coli,* in May, 1 000.

The figures for bacteria computed according to Mr. Hoskins' formula and those actually occurring are as given in Table 17.

TABLE 17.—Comparison of Computed and Actual Number of Bacteria.

	Total Bacteria.		*B. coli.*	
	Computed.	Actual.	Computed.	Actual.
Rockville, Conn	938 000	877 000	19 100	17 600
Manchester, Conn	247 000	246 000	5 040	7 400
South Manchester, Conn	70 042	150 000	12 400	2 200
East Hartford, Conn	469 000	102 000	9 543	396

The seasonal variation compares more favorably with results on the Ohio and Illinois Rivers, and there is a fairly close agreement between the computed and the actual numbers of bacteria below Rockville and Manchester both of which have sewage disposal plants which, however, are in very poor condition. Below South Manchester and East Hartford the results vary considerably. South Manchester has a sewage disposal plant, but it is not in operation. The dye wastes discharged from the silk mills may have an inhibiting action on the bacteria. Opportunity for sedimentation in the pond above the sampling stations may be another explanation. The samples were collected above the sewer outfall at East Hartford, where the river is affected by tidal action, but this would not explain entirely the differences obtained.

While Mr. Hoskins has evidently established a relation between sewage population and *B. coli* in streams receiving wastes, the general application

of his formula is apt to give quite misleading results. It is certain that it does not apply to streams of the character of the Naugatuck River, and its application to a stream similar to the Hockanum would lead to wrong conclusions unless one were familiar with conditions likely to effect its application.

The advisability of using the numbers of bacteria as an index where results are so erratic is questionable. At the best, the count of bacteria is only an estimate and the chance for error by using the prevailing technique is very great. Further, the magnitude of the number entering into the argument is so great that the variations between different places are relative only and the actual differences are hardly comprehensible.

Mr. Hoskins is to be complimented on the painstaking manner in which he attacked the problem. The practical adaptability of this method to all classes of streams, however, is doubtful.

MOMENTS IN RESTRAINED AND CONTINUOUS BEAMS BY THE METHOD OF CONJUGATE POINTS

Discussion*

BY F. E. RICHART, ASSOC. M. AM. SOC. C. E.

F. E. RICHART,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The method of analysis of indeterminate beams and frames given in this paper is excellent. The easy and rapid solutions presented should be welcomed by engineers as a means of bringing about a greater familiarity with, and consequently a greater use of, such structures. It appears, however, since no reference was made to it, that the authors were not aware of a method of graphical analysis quite similar to the one presented. Many of the essential constructions presented in this paper were published in 1883 by Fidler;§ others were added by Ostenfeld|| in 1913, or before.

Fidler in his original paper utilized the same properties of the simple-beam moment diagrams for the various spans of a continuous beam; provided for the location of the points, U_1, V_1, etc. (called by Fidler "characteristic points"), on the vertical through the one-third points of spans; fixed the location of the T-points on a line between adjacent U and V-points by the transposition of the one-third span lengths; and provided a means of locating the points here designated conjugate points and of drawing the negative moment lines, or M-lines, which constitute the solution of the problem. Fidler also devised constructions for the cases of unsymmetrical loadings, variations in depth of member along a span, and settlement of supports. He emphasized a physical concept of the construction that is not used by the authors, namely, that the distance from the characteristic points, U_1, V_1, to the M-line is proportional to the slopes or angular displacements of the elastic curve at the respective adjacent supports. This viewpoint is of particular value when the slope is known from conditions of restraint or of symmetry of loading.

Ostenfeld's graphical construction (corresponding to the "pennant diagrams" of the paper) replaced a simple calculation in Fidler's method by which the authors' "conjugate points", were located. Both the Fidler and

* This discussion (of the paper by L. H. Nishkian and D. B. Steinman, Members, Am. Soc. C. E., published in October, 1925, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Research Asst. Prof., Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, October 16, 1925.

§ *Minutes of Proceedings*, Inst. C. E., Vol. LXXIV, 1883, p. 196, and "A Practical Treatise on Bridge Construction", by Thomas Claxton Fidler, Fourth Edition, Lond., 1909.

|| "Teknisk Statik", by A. Ostenfeld, Vol. II, Second Edition, Copenhagen, 1913. Professor Ostenfeld added certain graphical constructions to Fidler's method in making a very general application to various forms of continuous beams.

Ostenfeld constructions were described by the writer some years ago in an article* wherein the method was extended to the analysis of a single column-and-girder frame, a hinged portal frame, and a frame consisting of four columns and three girders similar to that of Fig. 18.† For the latter frame the method of replacing the several unloaded members meeting at a joint by a single "equivalent" member (designated by the authors as the "Composition of Rigidities"), and an equation for the length of the equivalent member, were given. The modification of the method for unequal moments of inertia by "transformed spans" was also shown, as well as a treatment of members having various degrees of restraint at the ends.

Since the analysis by Fidler and Ostenfeld, although derived from a different viewpoint, arrived at constructions practically identical with those of the authors, it may be of interest to consider the reasoning used in Fidler's original demonstration. (Fidler made use of the effective depth of the girder rather than of the moment of inertia, but the substitution of the latter by Ostenfeld was an obvious step.) For convenience, the notation of the paper will be used wherever possible.

Consider, first, the simple beam of Fig. 32, having constant values of E and I and subject to two concentrated loads. The centroid of the moment diagram is at a distance, gl, from the end, R_1, and the area of the moment diagram is A. The slopes of the elastic curve at R_1 and R_2 are denoted by θ_1 and θ_2, respectively. By the well-known theorem of area-moments the deflection at R_1 of the elastic curve from the line drawn tangent to the curve at R_2 is $l\,\theta_2$ and is equal to the statical moment of the $\dfrac{M}{E\,I}$-diagram about R_1, or,

$$l\,\theta_2 = \frac{A\,g\,l}{E\,I}$$

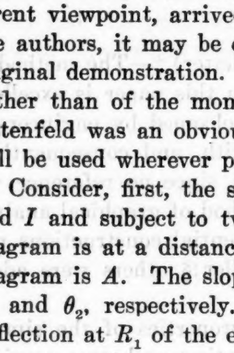

FIG. 32.

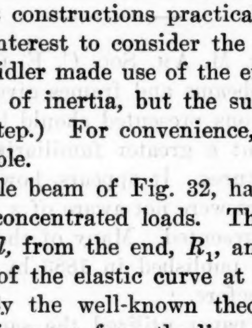

FIG. 33.

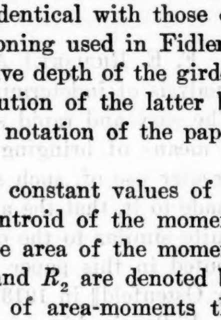

FIG. 34.

Now, consider the beam of Fig. 33 which is the same as that of Fig. 32, but is acted upon by external moments at the ends instead of by transverse loads. The moment diagram is a trapezoid varying in height from M_1 at

* "Graphical and Mechanical Analyses of Frames", by F. E. Richart and W. M. Wilson, *Engineering and Contracting*, Vol. 55, June, 1920, p. 700.

† *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1608.

R_1 to M_2 at R_2. Again, the deflection $l\,\theta_2$ at R_1 is equal to the statical moment of the $\frac{M}{E\,I}$-diagram about R_1, or,

$$l\,\theta_2 = -\frac{M_1\,l^2}{6\,E\,I} - \frac{M_2\,l^2}{3\,E\,I}$$

In Fig. 34, the same beam is shown subjected simultaneously to the concentrated loads and end moments of both preceding examples; in other words, it represents a typical span of a continuous beam. The moment diagrams of Figs. 32 and 33, when superimposed, or added algebraically, produce the moment diagram shown in Fig. 34. From the foregoing theory, the deflection at R_1 is:

$$l\,\theta_2 = \frac{A\,g\,l}{E\,I} - \frac{M_1\,l^2}{6\,E\,I} - \frac{M_2\,l^2}{3\,E\,I} \qquad (17)$$

and the slope at R_2 multiplied by $\frac{2\,E\,I}{l}$ is:

$$\frac{2\,E\,I}{l}\,\theta_2 = \frac{2\,A\,g}{l} - \frac{M_1}{3} - \frac{2\,M_2}{3} \qquad (18)$$

In a similar way it may be shown that,

$$\frac{2\,E\,I}{l}\,\theta_1 = \frac{2\,A\,(1-g)}{l} - \frac{M_2}{3} - \frac{2\,M_1}{3} \qquad (19)$$

In Fidler's construction the span is divided into three equal parts (Fig. 34) and ordinates erected at the points, U and V, making $U\,U_1$ equal to $\frac{2\,A\,(1-g)}{l}$ and $V\,V_1$ equal to $\frac{2\,A\,g}{l}$. From Equations (18) and (19) it is seen that the distances, $a\,U_1$ and $b\,V_1$, are thus made equal to $\frac{2\,E\,I}{l}\,\theta_1$ and $\frac{2\,E\,I}{l}\,\theta_2$, respectively. The points, U_1 and V_1, are called characteristic points and are the U and V-points of the paper, for the case of unsymmetrical loading shown in Fig. 14.* For symmetrical loading g is equal to $\frac{1}{2}$ and the ordinates both become equal to $\frac{A}{l}$, as is also shown in the paper. Since the distances, aU_1 and bV_1, are proportional to the slopes of the elastic curve at their respective adjacent supports, it is seen that, if the beam is fixed at R_1 and R_2, these distances are zero. If a partial restraint is offered at R_1 and R_2, the slopes at these points are inclined downward toward mid-span and the points, a and b, fall below U_1 and V_1. Conversely, if the end moments are sufficient to produce slopes at A and B that are inclined upward toward mid-span, a and b will lie above the characteristic points, U_1 and V_1.

Now, referring to Fig. 35, since adjacent spans of a continuous beam have the same slope at their common supports, it follows that the distances, aV_1 and bU_2, vary directly as the respective values of $\frac{E\,I}{l}$ for the two spans. If $\frac{E\,I}{l}$ is constant, aV_1 and bU_2 are equal and the closing M-line passes as far

* *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1603.

below the characteristic point, V_1, in one span as it does above the corresponding point, U_2, in the next span. If only EI is constant, aV_1 and bU_2 vary inversely as the values of l_1 and l_2, respectively. Fidler showed that with these limiting conditions the location of the M-line might be made by "cut-and-try" methods; but he also gave the following direct method of construction: The three-span continuous beam of Fig. 35 is considered to have a constant value of EI throughout with ends freely supported at R_1 and R_4. The simple-beam moment diagram for each span is first drawn and the characteristic points, U_1, V_1, etc., are located. The characteristic points, V_1, U_2 and V_2, U_3, are joined by straight lines and the points, T_{12} and T_{23}, located by transposing the one-third span distances.

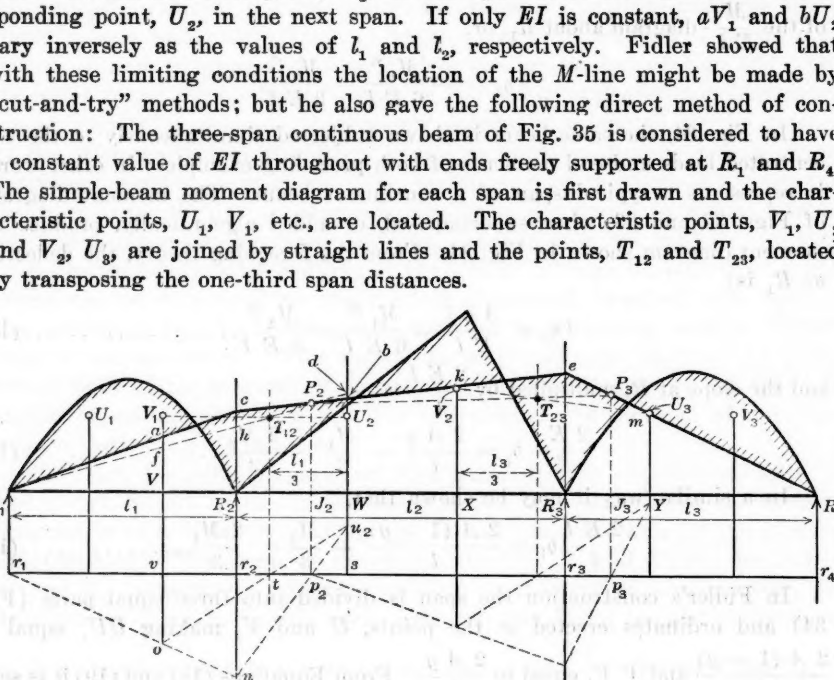

FIG. 35.

The M-line is now to be drawn so that $aV_1 \times l_1$ equals $bU_2 \times l_2$; this is done by properly locating the points, P_2 and P_3, on the M-line. If the distance, $R_2 W$, is divided so that $\frac{W J_2}{R_2 J_2} = \frac{R_1 V}{R_2 R_3}$, and the point, P_2, is located vertically above J_2 on the line, $R_1 T_{12}$, produced, then P_2 will lie on the required M-line. For, assuming the required M-line, cP_2b, to be correctly located in the diagram, it follows that:

$$a\,f = \frac{R_1\,V}{R_1\,R_2}\,c\,h = \frac{R_1\,V}{R_1\,R_2}\,\frac{R_2\,R_3}{R_1\,V}\,b\,d = \frac{R_2\,R_3}{R_1\,R_2}\,b\,d$$

and

$$V_1\,f = U_2\,d\,\frac{R_2\,R_3}{R_1\,R_2}$$

whence,

$$a\,V_1 = V_1\,f - a\,f = \frac{R_2\,R_3}{R_1\,R_2}\,(U_2\,d - b\,d)$$

therefore,

$$a\,V_1 \times l_1 = b\,U_2 \times l_2$$

Proceeding to the next span the distance, R_3Y, must be divided so that $\frac{Y\,J_3}{R_3\,J_3} = \frac{J_2\,X}{J_2\,R_3} \times \frac{l_2}{l_3}$. With J_3 thus located, the point, P_3, will lie at the inter-

section of a vertical through J_3 and the line, P_2T_{23}, produced. By a proof similar to that of the last paragraph, it may be shown that this construction will result in making the distance, $V_2k \times l_2 = U_3m \times l_3$. This satisfies the requirement that the distance from the M-line to the characteristic points, V_2 and U_3, shall be inversely proportional to the respective span lengths.

There are now two known points on the M-line in the third span and one in each of the first and second spans, so that the M-line may be drawn, starting from R_4, through P_3 to e, thence through P_2 to c and from c to R_1. The resultant moment diagram is represented by the area between the simple beam moment diagram and the M-line.

Obviously, the method might be applied to a beam having more than three spans by a repetition of the typical construction used in locating the point, J_3, thus locating a similar point, J_n, in each span.

It is seen that Fidler's method requires the division of the distances, R_2W and R_3Y, into two parts according to certain fixed proportions. A graphical method of accomplishing this division was devised by Ostenfeld.* Referring again to Fig. 35, it is required that the distance, R_2W, be divided so that $\frac{W\,J_2}{R_2\,J_2} = \frac{R_1\,V}{R_2\,R_3}$. It is convenient to draw this construction entirely separate from the moment diagram as shown in the lower part of Fig. 35, all vertical R, U, V, and T-lines being produced to intersect the line, r_1r_4. Starting at r_1 any line, r_1n, is drawn to intersect the R_2-line. From the point, o, a second line through t intersects the U_2-line at u_2, and the third line, u_2n, cuts the base line at p_2, which, when projected vertically upward to meet the line, R_1T_{12}, locates the point, P_2, as desired.

The proof of the construction is as follows:

$$\frac{W\,J_2}{R_2\,J_2} = \frac{u_2\,s}{r_2\,n} = \frac{u_2\,s}{o\,v} \times \frac{R_1\,V}{R_1\,R_2} = \frac{R_1\,R_2}{R_2\,R_3} \times \frac{R_1\,V}{R_1\,R_2} = \frac{R_1\,V}{R_2\,R_3}$$

Similarly, starting from p_2, which is the projection of a known point on the M-line, a second pennant diagram is constructed to locate the point, p_3, which, when projected vertically upward to meet the line, P_2T_{23}, locates the point, P_3.

It is seen that Ostenfeld's construction is identical with the "pennant diagram" of Fig. 9†. The construction has not been interwoven with the other construction lines of the moment diagram as in the paper, and the writer feels that the separate diagram is preferable in many ways. Since it is independent of the distribution of loading, the Ostenfeld diagram may be drawn entirely as soon as the R, U, V, and T-lines are drawn. The construction can be checked by drawing a second diagram beginning at the right-hand end of the beam.

It has been noted that Fidler's derivations included applications of the method (similar to, but less detailed than, those of the paper) to the cases of settlement of supports and of a variation of moment of inertia along the beam. It is clear that applications may also be made for various conditions

* "Teknisk Statik", Vol. II, Second Edition, Copenhagen, 1913.

† *Proceedings*, Am. Soc. C. E., October, 1925, Papers and Discussions, p. 1598.

of restraint of the extreme ends of the continuous beam. If the ends of a beam are fixed as in Fig. 36, the distance from the adjacent characteristic point, U_1, to the M-line, which is equal to $\frac{2\,E\,I}{l_1}$ times the end slope, is obviously zero, and the M-line goes through the point, U_1. It also follows that, since U_1 becomes the known point on the M-line, that the Ostenfeld diagram also begins at the U_1-line instead of at the reaction line through R_1.

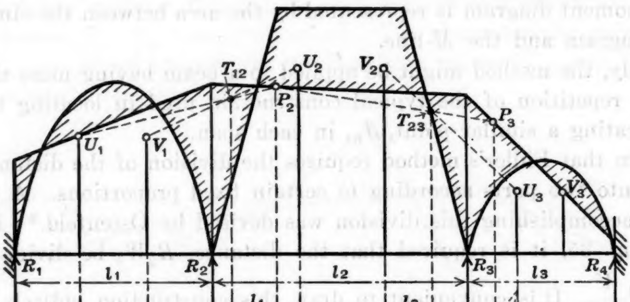

FIG. 36.

For the case of a known partial restraint of the ends, as might be the case of a test beam on which observations of end slopes have been made, the distances from the M-line to the U and V-points adjacent to the ends of the beam may be calculated. Fig. 37 shows such a beam, for which the end slopes are θ_1, and θ_4. The distances, $U_1 U$ and $V_3 V$, are equal, respectively, to $\frac{2\,E\,I\,\theta_1}{l_1}$ and $\frac{2\,E\,I\,\theta_4}{l_3}$, and, as the slopes indicate a degree of restraint less than fixity, the M-line will pass below the points, U_1 and V_3. This beam also illustrates the variation in construction for the case of an unloaded span. The characteristic points for the second span fall upon the base line, $R_1 R_4$, and all other lines are drawn in the regular way. Since the known points on the M-line are directly below U_1 and V_3, the Ostenfeld diagram is started vertically below one or the other of these points as shown.

If the restraint at the end is due to a known moment such as that due to a cantilever bracket or projection, the M-line may obviously start with a negative cantilever moment at the first reaction, R_1, and similarly the Ostenfeld diagram will start from the reaction line, R_1. The construction is illustrated in Fig. 38.

It is interesting that analyses proceeding from two distinct viewpoints should result in graphical constructions so nearly identical. The authors are to be commended on the treatment of secondary stresses and of variable moments of inertia of members. The tabulation of constants for beams having haunches of various shapes and sizes places the results of a large amount of analysis in a convenient form for reference, and gives a kind of information

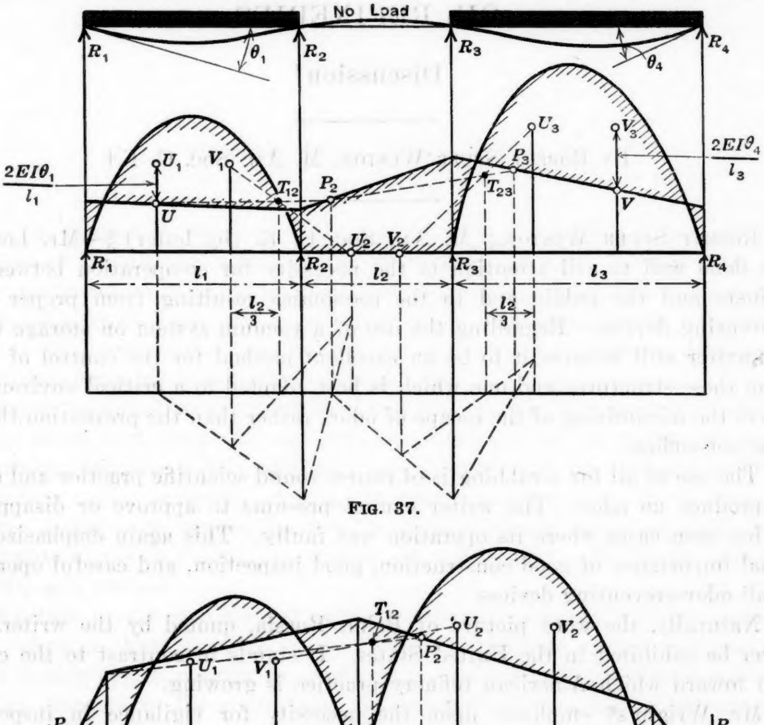

FIG. 37.

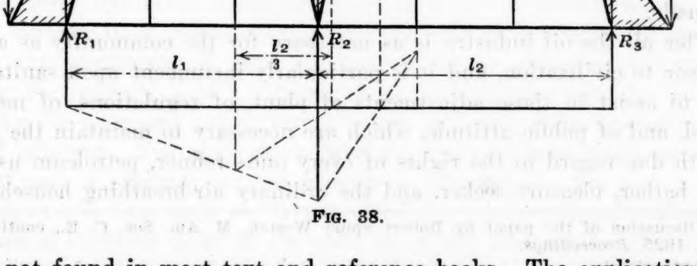

FIG. 38.

that is not found in most text and reference books. The application of the concept of conjugate points to continuous beam analysis gives a new and apparently useful line of attack on the problem, and the whole paper should be of distinct value in giving publicity to a rapid and simple solution of this particular class of statically indeterminate structures.

THE DETECTION AND ELIMINATION OF ODORS FROM OIL REFINERIES

Discussion*

BY ROBERT SPURR WESTON, M. AM. SOC. C. E.†

ROBERT SPURR WESTON,‡ M. AM. SOC. C. E. (by letter).§—Mr. Loomis‖ has done well to call attention to the necessity for co-operation between oil refiners and the public and to the economies resulting from proper odor-preventing devices. Regarding the use of a vacuum system on storage tanks, the writer still believes it to be an excellent method for the control of odors from these structures and one which is best adapted to a critical environment where the minimizing of the escape of odors rather than the prevention thereof does not suffice.

The use of oil for scrubbing is of course sound scientific practice and ought to produce no odor. The writer cannot presume to approve or disapprove; he has seen cases where its operation was faulty. This again emphasizes the equal importance of good construction, good inspection, and careful operation of all odor-preventing devices.

Naturally, the word picture of Baku, Russia, quoted by the writer, will never be exhibited in the United States. It stands in contrast to the condition toward which American refinery practice is growing.

Mr. Wright's¶ emphasis upon the necessity for vigilance in inspecting instruments, screens, traps, and other devices so subject to corrosion, is a point well made.

After all the oil industry is as necessary for the community as any other purveyor to civilization, and it is particularly incumbent upon sanitary engineers to assist in those adjustments of plant, of regulations, of methods of control, and of public attitude, which are necessary to maintain the rights of all with due regard to the rights of every one—refiner, petroleum user, navigator, bather, pleasure seeker, and the ordinary air-breathing householder.

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* Discussion of the paper by Robert Spurr Weston, M. Am. Soc. C. E., continued from August, 1925, *Proceedings*.

† Author's clo.

‡ Cons. Engr. (Weston & Sampson), Boston, Mass.

§ Received by the Secretary, October 9, 1925.

‖ *Proceedings*, Am. Soc. C. E., August, 1925, Papers and Discussions, p. 1193.

¶ *Loc. cit.*, p. 1195.

EXCESS CONDEMNATION IN CITY PLANNING

A SYMPOSIUM

Discussion*

By M. W. Weir, M. Am. Soc. C. E.

M. W. Weir,† M. Am. Soc. C. E. (by letter).‡—In excess condemnation the element of speculation looms large with the profit going to the deserving, that is, to the community in which an improvement requiring condemnation takes place. No doubt some will think that such a proceeding comes dangerously near to the acceptance of some of the political theories of several decades ago when the single tax was being discussed.

Others will look upon excess condemnation as a public usurpation of individual rights and the public ownership question will again be brought up for discussion, because public ownership of utilities will be confused with the temporary public ownership of property corollary to a public utility, such as a street. There is no ground tenable for questioning the policy of subjugating the rights of the individual, no matter what they are, to the public necessity, benefit, and welfare.

Whether or not a discarded political theory is revived is less a question than whether there is value in the project. Applying excess condemnation has a double object, namely, to eliminate an improbable individual speculation and to conserve to the community that accretion of values which the community as a whole creates when a public improvement is completed.

There never has been displayed an organization of individuals as parcel property owners in a community, with sufficient public spirit or sufficient trust in the worthiness of each other to carry on property improvements involving the re-plotting of a number of small holdings to fit the new lines created by the necessary public improvement affecting those properties abutting and adjacent. Few individuals have the financial resources necessary to carry out such a program nor the courage to attempt it. Yet such a public improvement will have only partly succeeded if it leaves ragged outlines and uneconomical building units in its wake, as in the cases of Lafayette Street and the Seventh Avenue Extension, New York City, cited in Mr. Leavitt's paper,§ and the Flatbush Avenue Extension in Brooklyn.

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* Discussion continued from November, 1925, *Proceedings*.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, October 29, 1925.

§ *Proceedings*, Am. Soc. C. E., September, 1925, Papers and Discussions, p. 1425.

In these three cases, certain communities or districts of the city have been the sufferers to the advantage of other districts lying to the north and to the south; all are glaring examples of the economic loss to the city at large in its failure to clean up the path and save to itself the increment of valuations of its own creation, now lying waste and impossible of individual development through many years.

It would even seem reasonable that adjoining owners had good grounds on which to sue for contingent damages because they could not share in the accretion of value which should be theirs and for which they had paid a share in the taxes and assessments.

Sins of commission are seized upon with avidity by political parties for campaign material; the misappropriation of petty hundreds of dollars receives glaring double headers in the newspapers and the story continues with gloating and glee, but the obscure sin of omission involving lost millions in assessed valuations, in such instances as Lafayette Street and the Seventh Avenue Extension, pass by unnoticed by political parties and unheralded by the press.

Some may say that with political corruption a constant evil, how much greater the opportunity for corruption in applying excess condemnation in public improvements. This may be true, but no one stops raising wheat and more wheat because rats in the granaries cause losses. Not long ago there appeared an article written by a manufacturer on the subject of city planning. Among other things it was stated that engineers, as a rule, permit themselves to become the victims of concentration and, therefore, to fall into narrowness of vision, failing to take a broad interest in public affairs.

This is not the first time such an accusation against engineers has been made, nor is it an unwarranted or undeserved statement. It is a regrettable truth that, singly, collectively, or as a professional class, engineers do not take the interest in public affairs that they should. Why is it; is it fear of criticism? Physical dangers galore have been encountered by engineers. It cannot be possible that they lack moral courage. Then let them prove that public questions, such as excess condemnation, have received their attention and let them demand that the public authorities put into practice those theories which engineers have discussed, weighed, and found worthy. In like manner, let them fearlessly condemn such sins of omission wherever they may be found.

MEMOIRS OF DECEASED MEMBERS

Note.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES EDWARD LAW BALDWIN DAVIS, M. Am. Soc. C. E.*

Died June 4, 1925.

Charles Edward Law Baldwin Davis, the son of Charles S. A. and Mary Jeanette (Downs) Davis, was born at New Haven, Conn., on February 16, 1844. After spending one year in the Academic Department of Yale College in 1861-62, he was appointed a Cadet at the United States Military Academy, at West Point, N. Y., on July 1, 1862. He was graduated from West Point on June 18, 1866, and commissioned a Second Lieutenant of the Corps of Engineers, United States Army. He was promoted to the rank of First Lieutenant, Corps of Engineers, on March 7, 1867; Captain on September 12, 1877; Major on April 7, 1888; Lieutenant-Colonel on May 3, 1901; Colonel on October 10, 1905; and Brigadier-General U. S. Army, on January 29, 1908.

Lieutenant Davis served on various military duties at Willets' Point and West Point, N. Y., and as Assistant Engineer in the construction of fortifications at Boston, Mass., Harbor, from 1866 to 1872. He was Assistant Engineer on the improvement of the mouth of the Mississippi River, from December 9, 1872, to May 1, 1876, and of Galveston Harbor, Texas, from 1876 to 1881. Captain Davis was then assigned to duty with the Light House Establishment, as Engineer of the Tenth Lighthouse District (1882), and afterward of the Eleventh District (1882 to 1886), and was also in charge of the improvements of the rivers and harbors of the Milwaukee, Wis., Engineer District, from 1882 to 1892.

From 1892 to 1896, Major Davis was on duty at Washington, D. C., in charge of the defensive works of the Potomac River, of certain river and harbor improvements in Maryland and Virginia, of the construction of a sewerage system at Fort Monroe, of the reconstruction of the Washington end of Long Bridge, of repairs to the Aqueduct Bridge across the Potomac, and of a survey and plan of a bridge across the Eastern Branch of the Potomac at the foot of South Capitol Street, Washington. He was also, during this period, a member of the Board of Officers to report on the various types of disappearing carriages for seacoast fortifications, and, from 1895 to 1896, was in charge of increasing the water supply of Washington, D. C.

From 1896 to 1902, Colonel Davis was stationed at San Francisco, Calif., in charge of the fortifications in that locality and in the Hawaiian Islands, as Engineer of the Twelth Lighthouse District. He also served as member of the California Débris Commission to regulate hydraulic mining. During part of this time he had charge of the improvements of the harbors of Wilmington, San Diego and San Luis Obispo, Calif., as well as of the construction of the breakwater at San Pedro, Calif.

* Memoir prepared by C. McD. Townsend, Col., U. S. A. (*Retired*), M. Am. Soc. C. E.

From 1902 to 1904 he was Division Engineer of the Philippine Division, and was in charge of the designing of fortifications and of military map-making in the Philippine Islands. From 1904, until his retirement from active service, on February 16, 1908, General Davis was in charge of the improvement of the channels connecting the waters of the Great Lakes, between Chicago, Ill., Duluth, Minn., and Buffalo, N. Y., his work including the design and commencement of the construction of the third lock in the canal at Sault Ste. Marie, Mich., the enlargement of the channel of St. Marys River, and the construction of the Livingston Channel.

In 1907 and 1908, he was also Division Engineer of the Lakes Division, Detroit River. After his retirement from the military service, General Davis was employed in 1910 and 1911, as United States Agent in charge of fortifications and river and harbor work in the San Francisco District.

General Davis had a most pleasing personality, and, although he never posed as a specialist in any particular branch of engineering, he had executive ability which enabled him to accomplish successfully the numerous and varied duties to which he was assigned during his professional career.

His fame as an engineer will be perpetuated by the inauguration of the lock which bears his name at Sault Ste. Marie. The Davis Lock is a concrete structure, 1 350 ft. long, between miter sills 80 ft. wide, and has 24½ ft. of water on its miter sills at low water, with a lift varying from 17 to 21 ft., dependent on the varying stages. With its companion, the Sabine Lock, which has recently been constructed, it is one of the largest lift locks in the world, being exceeded only by those of the Panama Canal.

General Davis was elected a Member of the American Society of Civil Engineers on July 12, 1877. He was also a member of the Army and Navy Club of Washington, D. C.

GEORGE DEVIN, M. Am. Soc. C. E.*

Died May 28, 1925.

George Devin, the son of M. L. and Amanda J. Devin, was born in Macon County, Illinois, on February 27, 1847. The family afterward removed to a farm near Des Moines, Iowa, where he attended the public schools.

In June, 1864, Mr. Devin enlisted in Company B, 48th Regiment, Iowa Infantry, and served until he was mustered out in the following October. In 1871, he entered Cornell University, from which he was graduated in 1873, with the degree of Bachelor in Civil Engineering.

Following his graduation, Mr. Devin was employed on construction work by the firm of T. B. White and Sons, of New Brighton, Pa. (afterward the Penn Bridge Company, of Beaver Falls, Pa.), and served as Superintendent of Shops for that firm.

He was afterward a member of the firm, and Manager, of the Pittsburgh Bridge Company, of Pittsburgh, Pa. Among his associates in the organization of this Company was Mr. John A. Nichols, formerly Master Mechanic of the

* Memoir prepared by T. Sidney White, M. Am. Soc. C. E.

Keystone Bridge Company. One of the notable works of that firm was the reconstruction of the Railway Suspension Bridge at Niagara Falls. In 1881, he engaged in independent contracting for bridges in connection with the Delaware Bridge Company, of Athens, Pa., of which Charles Macdonald, Past-President, Am. Soc. C. E., was then President.

During practically all his subsequent work, Mr. Devin was engaged in the design and construction of bridges, in contracting in connection therewith, or in a consulting capacity. He was identified at different times with the Wrought Iron Bridge Company, of Canton, Ohio, as Assistant Superintendent; the King Bridge Company, of Cleveland, Ohio, as Engineer; the Atlanta Bridge Company, of Atlanta, Ga., as Superintendent of Erection; with the late Theodore Cooper, M. Am. Soc. C. E., of New York, N. Y.; the Pottsville Bridge Company, of Pottsville, Pa., as Chief Engineer; the Penn Bridge Company, of Beaver Falls, Pa., the American Bridge Company, the Kansas City Terminal Company, Kansas City, Mo., etc. He also was engaged in private practice as a Consulting Engineer in Chicago, Ill., New York, N. Y., and St. Louis, Mo.

On July 25, 1876, Mr. Devin was married to Emma R. Lowry, of New Brighton, Pa., who died in St. Louis, Mo., in 1922.

Shortly after his wife's death, Mr. Devin retired from active work and, later, became an inmate of the National Home for Disabled Volunteer Soldiers at Sawtelle, Calif., where he remained until his death. He was buried in the Home Cemetery with full military honors on June 1, 1925.

He was the eldest of six brothers, only one of whom, James Devin, of Des Moines, Iowa, and a sister, Mrs. Sarah D. Clarke, of Redwood City, Calif., survive him.

Mr. Devin had a great liking for mechanical problems, and took much pleasure in mathematical calculations, investigations, and analyses, and, also, in studies of stresses in iron and steel structures.

He was a man of absolute integrity, considerate of others, and possessed of a strong sense of loyalty to and affection for his friends.

Mr. Devin was elected a Member of the American Society of Civil Engineers on September 7, 1887.

CHARLES WELLS EDDY, M. Am. Soc. C. E.*

Died June 18, 1925.

Charles Wells Eddy was born on November 8, 1875, at Brunswick, N. Y. He was of Pilgrim stock, having been a descendant of Samuel Eddy, who came to America in 1630, and Kenelm Winslow, who came in 1629, both joining the Pilgrim Company at Plymouth, Mass. He was also a descendant of Robert Collins, of Sandown, N. H., who served as Captain of the New Hampshire Volunteers in the Revolutionary War.

In 1884, Mr. Eddy moved with his parents to Simsbury, Conn., where he received his early education at the public schools and McLean Seminary. He

* Memoir prepared by R. A. Cairns, M. Am. Soc. C. E.

then entered the Connecticut Agricultural College and was graduated in the Class of 1893.

From 1893 to 1900, he was engaged in land surveying and in surveying for and inspection of State roads for various towns of Connecticut. From November 14, 1900, until his death, on June 18, 1925, with the exception of about a year, Mr. Eddy was employed by the City of Waterbury, Conn., in the Bureau of Engineering. During 1900 and 1902, he was engaged as Surveyor on the completion of the Wigwam Reservoir of the Waterbury Water Supply. In 1906 and 1907 he served as Resident Engineer in charge of the construction of the dam of the Coe Brass Company, at Torrington, Conn.

From 1908 to 1916, Mr. Eddy was engaged in preliminary surveys for, and was Resident Engineer in charge of, the construction of a dam for the Morris Reservoir which forms a 2 000 000 000-gal. storage reservoir for the City of Waterbury. Following this engagement he was employed in other water supply work and in making preliminary surveys for the 7-mile tunnel which the City of Waterbury is now constructing to enlarge its present water system, and also in designs for future controlling works for this system. At the time of his death, he was Resident Engineer at Thomaston, Conn.

Mr. Eddy was always loyal and keenly interested in his work, never sparing himself when it required unusual attention.

That he was willing to devote some of his time for the benefit of others is shown by the fact that besides being a member of the Society, he had also been a member of the Connecticut Society of Civil Engineers since January, 1900, having been its President at the time of his death. He was also a member of the New England Water Works Association, and of the American Society for Municipal Improvements; Director of the Thomaston Water Company; President of the Thomaston Sewer Company, the Thomaston Business Men's Association, and of the Masonic Temple Corporation of Thomaston.

Mr. Eddy was a member of the Congregational Church of Thomaston, and a Thirty-second Degree Mason.

He was married at Simsbury, Conn., on July 31, 1901, to Gertrude Holcombe, who survives him.

Mr. Eddy was elected an Associate Member of the American Society of Civil Engineers on March 2, 1909, and a Member on June 24, 1914.

WILBUR FRANCIS GOODRICH, M. Am. Soc. C. E.*

Died February 26, 1925.

Wilbur Francis Goodrich, the son of Charles A. and Eleanor M. Goodrich, was born on August 18, 1853, at Lunenburg, Mass. He was educated at the Cambridge, Mass., Grammar and High Schools and at Comers' Commercial College at Boston, Mass. In 1873, he entered the Massachusetts Agricultural College, but remained only one year, having been compelled to leave because of illness.

* Memoir compiled from information on file at the Headquarters of the Society.

On April 27, 1874, he began his engineering studies in the office of Shedd and Sawyer, Civil Engineers of Boston, where he remained for three years. His first work was that of surveying house lots at Chestnut Hill Reservoir, Newton, Mass. In 1877, he was employed on the Newton, Mass., Water-Works.

In 1880, Mr. Goodrich was engaged from June to September by the Nashua and Lowell Railroad Company as Assistant to the late Frederick Brooks, M. Am. Soc. C. E.; in September and October, he was with the late James B. Francis, Past-President, Am. Soc. C. E., on work on the Locks and Canals at Lowell, Mass., and in November and December, he was in the service of the Massachusetts Central Railroad Company as Assistant, in charge of a party, laying out work for construction.

From December, 1880, to March, 1881, Mr. Goodrich was employed by the City of Lawrence, Mass., as Transitman and Levelman on a survey of a section of the Valley of the Spicket River. From April, 1881, to January, 1883, he served the Fitchburg Railroad Company as Assistant in charge of a party, under the late Edmund K. Turner, M. Am. Soc. C. E., Chief Engineer, his principal work being the laying out of work for construction.

From January, 1883, to April, 1884, he was employed by the New York and New England Railroad Company as Assistant Engineer in charge of construction. This was contract work for which Mr. Goodrich prepared all the plans and made all the estimates. From April to December, 1884, he was with Whitman and Breck, Civil Engineers, of Boston.

From April to August, 1885, he served with the late William S. Barbour, M. Am. Soc. C. E., Chief Engineer of the City of Cambridge, Mass., as Inspector on the laying of 3 miles of 30-in., iron, water pipe and also in the removal and rebuilding of a drawbridge. From October to December, 1885, he was with the Massachusetts State Drainage Commission and for the six months following with Mr. E. Bowditch, of Boston, on topographical surveys at Ipswich, Mass.

In 1886 Mr. Goodrich was appointed Resident Engineer on the Toledo, St. Louis, and Kansas City Railroad, in charge of changing 63 miles of road from narrow to standard gauge and the building of a stone bridge foundation, a pier, and two abutments over the Big Wildcat River, two miles west of Kokomo, Ind. Following this engagement he was employed as Resident Engineer on the Mackinaw, Toledo, and Saginaw Railroad, and, subsequently, he was engaged for six months as Consulting Engineer on water-works in Vermont and on plans for electric street railways at Steubenville, Ohio, and Newport, R. I., as well as a reservoir for the United States Government on Coasters Harbor Island, Rhode Island.

As a Contractor with Mr. J. F. Barry, of New York, N. Y., Mr. Goodrich constructed three miles of electric railroad at Steubenville, one mile of street-railroad on Southern Boulevard, New York, and two miles at Richmond, Va. He also installed a 34-in. pipe line through ledge rock in Rockville, Conn. He then accepted the position of Resident Engineer with the Kansas City and Toledo Railroad Company, his work consisting of change of line covering 76 miles, from Ramsey, Ill., to St. Louis, Mo., including the construction of bridges, etc. Subsequently, he became Assistant Engineer with the Old Colony

Railroad Company at Boston, and, later, was engaged in private engineering work, including the construction of the sewer system for the Metropolitan Park Commission of Massachusetts and change of line of the Revere Beach Railroad at Crescent Beach, Revere, Mass. His work also included change of grade at street crossings at Lowell, Mass., for the Nashua and Lowell Railroad Company.

From 1891 to 1894, Mr. Goodrich served as Assistant Engineer of the Metropolitan Sewerage Commission of Massachusetts. During this period he also constructed a wharf at Deer Island, in Boston Harbor, and for ten months following this engagement he served as Construction Engineer of a reservoir for an additional water supply at Yarmouth, Mass. He also held the position of Consulting Engineer for the Portland Stoneware Company's Plant, at Portland, Me., and was engaged for six months on inspection work at Payson Park Reservoir, at Cambridge, Mass. He also acted as Consulting Engineer for the construction of a water-works system at Edwardsville, Ill., and for the Coffeen Coal and Copper Company, on the construction of its plant at Coffeen, Ill.

In July, 1896, Mr. Goodrich became Assistant Engineer of the Boston Terminal Company in charge of laying out foundations, tracks, etc., for the New South Station, Boston. In 1900, he served for six months with the New York Central and Hudson River Railroad Company at Buffalo, N. Y., and also as Consulting Engineer of coal and iron property at Bedford, Pa.

In 1901, he had charge of construction at the Stanley Electric Plant, at Pittsfield, Mass., and, at the same time, acted as Consulting Engineer for the Austin Street Railroad, at Austin, Tex. He later served as Consulting Engineer for the Westfield Water-Works, Westfield, N. J., and the Piqua, Ohio, Electric Street Railroad Company, and made surveys and estimates for an electric railway from Washington, D. C., to Great Falls and Fairfax Court House, Va.

In 1902, Mr. Goodrich acted as Consulting Engineer for mill works at Adams, Mass., and examined coal property at Riddlesburg, Pa., and Middlesboro, Ky. He was also engaged as Consulting Engineer for mills at Saugus and Lawrence, Mass.; for the electric street railway at North Andover, Mass.; the Revere Boulevard, Revere, Mass.; and made plans and estimates for the Montreal Terminal Water Power Plant at Kinderhook Creek, N. Y.

In the same year he was placed in full charge of the work at the plant of the Gale Creek Coal Company, at Wilkeson, Wash., which comprised opening the mine, installing new machinery, and constructing new buildings. He also made examinations and prepared reports on the Denver, Lakewood, and Golden Railroad. In the State of Washington, he reported on the water power of the Nesqually River, the coal property of the Burnett Coal Company at Carbonado, and coal mines at Franklin, Fairhaven, and Whatcom, as well as coal property at Fairfax and Mulmont. He also made an examination and reported on the Tacoma and Eastern Railroad at Tacoma.

In 1903 and 1904, Mr. Goodrich served as Engineer in charge of pumping out and installing machinery in a coal mine in Gayton, Va., and as Con-

sulting Engineer on work in other cities in New England and the South. He also served as Consulting Engineer and reported on the construction of a plant for the Amalgamate Graphite Company at Johnsburg, N. Y., and, in 1905, made examinations and reports on the Connecticut River below Springfield, Mass., as well as for the Newfoundland, Labrador, and Sandwich Bay Company.

From July, 1905, to July, 1906, he was engaged as Consulting Engineer for the Nevada Northern Railroad Company and from 1906 to 1908 in private work in Boston. In 1909, he was employed as Consulting Engineer to the Boston Finance Commission and, in 1910, was Engineer in charge of pumps and engines in mills at Adams, Mass. He also held the position of Engineer of the Municipal Filtration Company at Boston.

In 1911, he served as President of the T. M. Filter Company of Boston and, until 1915, carried on a private consulting practice in several cities in Massachusetts. In 1916, he was again Consulting Engineer for the Boston Finance Commission; was engaged in machine-shop designing at Fitchburg, Mass.; and served as Engineer of the Shirley Peat Works at Shirley, Mass.

In 1917, and 1918, Mr. Goodrich held the positions of Engineer for the Elm Street Bridge at Manchester, N. H.; Resident Engineer on the Fort Smith and Western Railroad; and Consulting Engineer for the Boston and Maine Railroad Company. During this period he was also Inspector of the building of a wharf, pier, and pier shed at the Army Base at South Boston, Mass., and, in 1919, he became Inspector at the Army Base and also held the position of Construction Engineer for the Eastern Massachusetts Street Railway Company, with work at Chelsea, Randolph, Salem, Peabody, Revere, Weymouth, Lynn, and Lowell, Mass.

In 1921, he was appointed Inspector of a bridge over the Connecticut River at Springfield, Mass., and from 1922 to 1923 was engaged in private work for the Boston Finance Commission and was also Engineer in charge of the relocation of railroad yards and tracks at Springfield, Mass.

From 1924 until his death on February 26, 1925, Mr. Goodrich was engaged in private practice which included the building of a coal trestle, water tanks, and Colonial filling stations for the J. F. Kennedy Company, of Boston.

He is survived by his wife, Mary L. Goodrich, two sons, and two daughters.

Mr. Goodrich was a member of, and Treasurer of, the Railway Appliance Association of Providence, R. I., and an Associate Member of the Alumni of the Massachusetts Agricultural College. He was a member of the First Methodist Episcopal Church and, also, of Soley Lodge, F. and A. M., of Somerville, Mass.

He was essentially a home man, taking more pleasure with his family than with social clubs and organizations. Having a kindly disposition and a ready smile he made friends easily and retained such friendships throughout his life.

Mr. Goodrich was elected a Member of the American Society of Civil Engineers on May 4, 1887. He was also a member of the Northeastern Section of the Society.

WILLIAM SIMPSON KELLER, M, Am. Soc. C. E.*

Died September 9, 1925.

William Simpson Keller was born, on February 20, 1874, at Tuscumbia, Ala., and was a son of Arthur Henley and Sally (Simpson) Keller. His father who was a native of Tuscumbia, was Captain and Paymaster in Roddy's Command, Confederate Army, was promoted to the rank of Major just as the war closed, and was a prominent lawyer and editor. He was a grandson of David and Mary (Moore) Keller, of Tuscumbia, the latter a direct descendant of Governor Spottswood, first Colonial Governor of Virginia, and a second cousin to General Robert E. Lee.

Mr. Keller received his early schooling at Tuscumbia, attended the State Normal School at Florence, Ala., and was graduated with the degree of Bachelor in Civil Engineering from the University of Alabama in 1893.

He was County Surveyor of Colbert County, Alabama, during 1897 and 1898; Assistant Engineer of Shiloh National Park from 1900 to 1905; Chief Engineer of the Good Roads Commission, Madison County, Tennessee, from 1905 to 1910; with the United States Office of Public Roads from 1908 to 1909; Engineer in charge of road construction in Dallas County, Alabama, in 1910 and 1911. In 1911, Mr. Keller was appointed State Highway Engineer of Alabama, which important post he held until his death.

The love, respect, and esteem in which Mr. Keller was held could not be better stated than in the following editorial that appeared in the *Montgomery Advertiser,* Montgomery, Ala., on September 10, 1925:

"Simpson Keller

"The death of William Simpson Keller, in the prime of his life, is a distinct loss to the people of Alabama, and to the State government which is engaged in administering the benefits to the people which they have claimed. Both the people and the State government are poorer by the death of this high-minded and efficient public servant who had demonstrated his capacity for a highly specialized public service.

"His friends, those who knew him well and intimately, find inexplicable the decree of Providence which removed from his place in the activities of mankind this efficient and faithful public servant who through many years of apprenticeship and experience had demonstrated that he possessed in a rare degree the ability and energy which his high professional position required. He was essentially a product of Alabama. He was born among the people of Alabama, lived among them all his life, was educated among them and among them did his work of life. The death of this man of ability and character recalls the life of his father, who was so well known and admired by the members of a past generation. Captain A. H. Keller was a member of the Civil War generation and he faithfully served the Confederacy as a soldier till the collapse of the Confederacy, which involved the people of the South in financial and material ruin. For many years afterwards Captain Keller was the brilliant editor of a paper in Tuscumbia, which exerted no little influence in the affairs of the State. There in clean-minded, high-thinking, his children grew up about him. It is a tribute to a fine example of an older generation merely to say that he was the father of that famous genius, Helen Keller, who cour-

* Memoir prepared by R. P. Boyd, M. Am. Soc. C. E.

ageously overcame the handicaps of Nature, and of Simpson Keller, whose expert professional skill inaugurated the era of good roads in Alabama.

"The younger Keller prepared for his professional career at the University of Alabama and in him was mingled the love and pride in his chosen profession and a love for and an appreciation of his native State. Alabama was preparing to enter upon an era of good road building, but the sentiment had not found adequate expression. The first stage of the campaign to give Alabama good roads was the bonding of several counties to finance the great undertaking in those counties. It was natural that Dallas, a leader of this movement of progress, should call in for direction a native of Alabama who had previously demonstrated his energy and ability, and so Simpson Keller was called in as County Engineer to undertake this great work. Much depended upon the integrity and professional skill of these early county engineers. The taxpayers had taxed themselves to pay for a great public work the nature of which they had not known, as no work of this kind had been done by previous generations, and they scrutinized closely this heavy expense and the results that it brought. Dallas County entered the work in a whole-hearted way, but somewhat timidly and nervously. But under the direction of Mr. Keller, this work of road building was an unqualified success and when the taxpayers of Dallas viewed and used the good roads they were eminently satisfied that they had authorized the bond issue. In time the creation of good roads became an activity of the State and the able Engineer who had been so successful in Dallas County, was called to superintend the greater work of building good roads for the State. He made his job one of the most important in the State, and although the law relating to the State Highway Commission was inadequate, this law was never altered without some of the men of influence considering its possible effect on Mr. Keller's office.

"As to how a road should be built and where it should be located, there are many differences of opinion among men. Through the many years he was in office, Mr. Keller was successful in composing these difficulties, not merely because he was diplomatic and polite, but because he was just and impartial and guided by the highest principles of integrity. Many of our States have had highway scandals over the misuse of the large sums of money involved in their construction, but there has been none in Alabama, where the funds were administered by an honest, high-minded gentleman, who strove always to administer the trust that was imposed in him for the good of the people. The work for good roads in Alabama is far-flung and comprehensive. Much of it has already been accomplished and in these splendid highways will be found the monument of Simpson Keller, who was the right man in the right place. It is the beginning of an era and a man who was conspicuous in ushering in the era will not be forgotten by a grateful people when they travel the smooth and beautiful highways which they owe in a large degree to his skill.

"Mr. Keller, in his dependability, his faithfulness to his duty, and his grasp of the problems of his office was an inspiration to his associates, both his superiors and his subordinates.

"He was a man of striking personality, warm, genial and cordial, and considerate of the rights of all. He leaves a most pleasant memory to all who knew him and tender and pleasant memories to those who were admitted to his friendship. A faithful public servant, a patriotic citizen, a loyal friend, and a helpful neighbor who loved his fellow men passed when he died. He went in the prime of life and when he had every reason to expect many years of useful activity. He died literally 'with his harness upon his back.'"

Mr. Keller was an Elder in the Presbyterian Church, a Rotarian, a Fellowcraft Mason, but was taken ill before receiving his master's degree. He served as Director and Past-President of the American Association of State Highway Officials, and was a Director of the American Road Builders Association.

In 1902, he was married to Aline Moore, of Columbia, Tenn., who died in February, 1911. In 1913, he was married to Annie Searcy, of Tuscaloosa, Ala., who survives him. Other relatives are two sisters, Mrs. L. Warren Tyson, of Montgomery, Ala., and Miss Helen Keller, of New York, N. Y.; one brother, Phillips B. Keller, of Mineral Wells, Tex., and an aunt, Mrs. L. K. Lasseter, of Selma, Ala.

Mr. Keller was elected a Member of the American Society of Civil Engineers on July 2, 1913.

JAMES ALBERT McDONOUGH, M. Am. Soc. C. E.*

DIED AUGUST 8, 1925.

James Albert McDonough was born at Knoxville, Tenn., on December 17, 1869, the son of Dr. James S. and Mary Lenorr McDonough.

He received his education in the schools of his native city, completing the course in Civil Engineering at the University of Tennessee and taking post-graduate work there in 1894 and 1895.

In June, 1895, Mr. McDonough entered the service of the Ohio River, Knoxville, and Tidewater Railway Company and between this date and 1897 was employed by that company and others on topographic surveying and drafting work. From 1897 to 1901, he was engaged as Inspector and Draftsman at Port Royal, S. C., on the design and construction of buildings, dry docks, and other naval yard works, and from 1901 to 1904, was employed at the Navy Yard at Boston, Mass., and the United States Bureau of Yards and Docks, Washington, D. C., as a Designer on similar work.

In November, 1904, he entered the United States Engineer Department at Wheeling, W. Va., as a Junior Engineer engaged on the design of locks, dams, etc., for the canalization of the Ohio River, and from this time was employed on this work in various capacities. He was promoted to be Assistant Engineer in 1914, and in 1915 was transferred from Wheeling to the office of the Division Engineer at Cincinnati, Ohio, where, at the time of his death, he was acting as Head of the Designing Department and Assistant to the Division Engineer.

Mr. McDonough was married in 1916 to Martha Longley, of St. Clairsville, Ohio, who with his mother and three sisters survives him.

He was deeply interested in his profession and devoted much of his time to its study and to participation in its activities. While well versed along broad engineering and scientific lines, Mr. McDonough possessed special knowledge of the inland waterways of America and Europe and was regarded as an authority on that subject. Always a firm believer in the ultimate success of the Ohio River improvement, on which he spent so many years of his life's work, his inability to see its approaching completion is particularly regrettable.

* Memoir prepared by C. W. Kutz, M. Am. Soc. C. E.

Mr. McDonough was elected an Associate Member of the American Society of Civil Engineers on April 3, 1907, and a Member on May 3, 1910. He recently served as President of the Cincinnati Section of the Society and was an active member of the Engineers Club of Cincinnati.

WILLIAM LEROY REYNOLDS, M. Am. Soc. C. E.*

Died April 16, 1925.

William LeRoy Reynolds, the son of Charles N. and Minnie C. Reynolds, was born at Dundas, Ohio, on April 26, 1886. He began his elementary schooling in McArthur, Ohio. In 1895, his parents moved West, settling in Colorado, first in La Junta, then in Monument, and, finally, in 1900 locating permanently in Denver, in which city Mr. Reynolds finished his public school education preparatory to entering college. He was graduated from the University of Colorado, at Boulder, in 1909, with the degree of Bachelor of Science in Civil Engineering.

Soon after his graduation he spent several months at Palisades, Colo., where he was engaged in both office and field work for the Orchard Construction Company. He next accepted the position of Office Engineer for the Antlers Orchard Development Company, at Silt, Colo., an irrigation system for 4 000 acres. From March until December, 1910, he served as Construction Engineer for the Glenwood Light and Power Company, and was in charge of the construction of nine miles of steel pole transmission lines, substations, power-house, and water-works improvements at and near Glenwood Springs, Colo. This work cost $40 000. From December until June, 1911, he made various estimates, surveys, and designs for the Antlers Orchard Development Company, the Glenwood Light and Power Company, the Colorado Orchard Development Company, and the United States Forest Service.

From June, 1911, to July, 1912, Mr. Reynolds was Division Engineer on the construction of Willcox Canal System covering 10 000 acres near Rifle, Colo. From August, 1912, to June, 1913, he was employed by the Board of Public Works, at Denver, as Assistant Engineer, in charge of the Cherry Creek reconstruction work, bridges, walls, sewers, etc., the aggregate cost of which was about $300 000.

In July, 1913, he was appointed Construction Engineer on the replacement of outlet works, at Big Horn Reservoir, near Sheridan, Wyo., the cost of which was estimated at $40 000. After its completion, in December, 1913, he was engaged in private work in Glenwood Springs and Denver, until April, 1914, when he became Assistant Engineer for the Public Utilities Commission of the City and County of Denver, on construction and water-rate cases. He completed this work in August, 1915, and immediately afterward accepted the position of Assistant Engineer for the Van Sant-Houghton Company, and was engaged in estimating the cost of a new water plant for the City of Denver.

In July, 1916, and for the next two years, Mr. Reynolds served in the capacity of Field Engineer for the American Smelting and Refining Company,

* Memoir prepared by M. C. Hinderlider, M. Am. Soc. C. E.

at Leadville, Denver, Pueblo, and Durango, Colo. During this time he was in charge of the design and erection of smelter structures worth about $350 000.

In March, 1918, he had the honor of being appointed by the Government to a position of great responsibility for the period of the World War. He was sent to Nitro, W. Va., where he was made Supervising Engineer, in the Office of the Director of the U. S. Explosives Plant C. Mr. Reynolds was in direct charge of most of the construction and, later, was given the management of the operation of the water system (capacity 60 000 000 gal. per day, the cost of which was about $2 000 000) and also of the sewer and gas systems.

His work at Nitro was valued very highly by the Government which showed its appreciation by appointing him Chairman of the Plant Salvage Board, at the signing of the Armistice. The confining work, long hours, and stern responsibilities during the War, however, had left their imprint upon him. His health began to fail and before the Salvage Board had finished its task of disposing of the plant, Mr. Reynolds was obliged to resign and return to Colorado. He established himself on a small fruit farm near Silt, in 1920, and for the greater part of the next four years devoted most of his energies toward regaining his health. During this period he could engage only in advisory work, and although he was elected President of the Farmers Irrigation Company, his activities were limited.

In April, 1924, he felt that his health had been sufficiently restored to enable him to accept an appointment from the State of Colorado, that of Hydraulic and Railway Engineer, with headquarters in Denver. He held this position for almost a year, when he suffered a physical relapse and, after a short illness, died at his home in Denver on April 16, 1925.

Mr. Reynolds was a member of the Capitol Heights Presbyterian Church, of Denver, and of the Glenwood Springs Lodge No. 65, Royal Arch Masons No. 22.

He was married to Luellen Comer, of Glenwood Springs, Colo., on May 5, 1912. Besides his wife, and daughter, his parents and a sister survive him.

Mr. Reynolds was elected an Associate Member of the American Society of Civil Engineers on February 4, 1914, and a Member on April 14, 1919.

JAMES TOWNSEND TAYLOR, M. Am. Soc. C. E.*

Died March 17, 1924.

James Townsend Taylor was born in Waiaha, Kona, Hawaii, on March 19, 1859. He was the son of the Rev. Townsend Elijah and Persis Goodale (Thurston) Taylor.

California having become the home of the family, he attended Golden Gate Academy, Washington College, and the University of California.

In 1877, Mr. Taylor began his engineering work with the South Pacific Coast Railroad Company and remained in its service until 1879. He was with the California State Engineering Department in 1880 and 1881; with the

* Memoir prepared by Charles H. Kluegel, M. Am. Soc. C. E.

Pomona Land and Water Company in 1883; and served as City Engineer of Pomona, Calif., in 1887 and 1888. Mr. Taylor was engaged in public and private practice in Southern California from 1889 to 1896, and was connected with work on an irrigation project in New Mexico from 1896 to 1898. From 1899 until his death he was engaged in public works and carried on a private practice in Hawaii.

He was a member of the Honolulu Chamber of Commerce, the Honolulu Commercial Club, the Independent Order of Odd Fellows at Pomona, Calif., and the American Association of Engineers.

He was married on October 31, 1894, to Ethel May Webster, who, with a son, Thurston W. Taylor, survives him.

Mr. Taylor was elected a Member of the American Society of Civil Engineers on January 3, 1894.

RAYMON EDGAR FULCHER, Assoc. M. Am. Soc. C. E.*

Died July 14, 1925.

Raymon Edgar Fulcher was born at Jackson, Calif., on August 4, 1879. His early education was received in the public schools and High School of Oakland, Calif. During vacations and at other times until he was nineteen years of age, he was engaged in and about the gold mines and ore-dressing plants of the Mother Lode District of California, with his uncle, Wilton E. Darrow, and it was after this experience that his leaning toward an engineering career developed and became the most prominent factor in his life's work. Mr. Fulcher was largely self-educated in engineering, having been a keen observer and a persistent reader and student of technical civil engineering literature.

In 1904, he passed an examination at Sacramento, Calif., and was appointed Deputy County Surveyor of Tuolumne County, California. He also served as Deputy United States Mineral Surveyor and was engaged in underground mine surveying on the Mother Lode Mines in the vicinity of Sonora, Calif.

Mr. Fulcher's work as an engineer was varied in character, embracing mining, railroad engineering, tunnel construction, road work, earth-fill dams, rock-fill dams with concrete face, high-tension transmission-line construction, pumping plants, electric car lines, and oil pipe-line construction. As a result of his experience in mining and kindred activities he examined and reported on many mining properties and oil prospects in California, and, later, in Arizona.

In 1905, Mr. Fulcher was engaged by Sanderson and Porter as Field and Construction Engineer on work for the Stanislaus Electric Power Company, in California, which included the location of a construction railroad, flume and dams, and new road work and transmission lines. In 1911, he built an electric car line between the towns of Raymond and Southbend, Wash., on Willapa Harbor. He continued with Sanderson and Porter on hydro-electric develop-

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* Memoir prepared by Wynne Meredith, M. Am. Soc. C. E.

ment construction in British Columbia and Vancouver and on a pumping plant at Burbank, Wash., until 1914, when he was appointed Field Engineer in charge of the construction of the Valley Oil Pipe Line, extending 170 miles from Coalinga to Martinez, Calif., on the Bay of San Francisco.

From 1916, until the summer of 1918, Mr. Fulcher was engaged in a similar capacity on the construction of an oil pipe line across the States of Oklahoma and Missouri, terminating on the east bank of the Mississippi River in Illinois, a few miles above St. Louis, Mo. The crowning achievement of his career as an Engineer was probably the laying of this pipe line across the Mississippi, which was an important link in supplying fuel oil to the Allied fleets during the latter period of the World War.

During this work, which was continued without interruption through the excessively cold winter of 1917-18, Mr. Fulcher contracted pneumonia, which made it necessary for him later to live in Tucson, Ariz. While there, he acted as Engineer in charge of the installation of a 1 000-h.p. Diesel engine electric unit for the Tucson Gas, Electric Light, and Power Company. He then engaged in building construction and general engineering practice, and, later, specialized in roofing and roof covering, developing a successful business of which he was in charge at the time of his death. He died on July 14, 1925, at Phœnix, Ariz., of bronchial pneumonia, caused by the extreme heat.

Mr. Fulcher was married on March 8, 1901, to Blanche E. Divoll, of Sonora, Calif. His wife and three daughters survive him.

As an engineer of ability, Mr. Fulcher combined with his technical attainments, an open mind, executive qualities, and a winning personality. To his more intimate friends he will be remembered for his modest ways, his fine sense of humor, and those rare qualities of mind and heart that so endeared him to all with whom he came in contact.

Mr. Fulcher was elected an Associate Member of the American Society of Civil Engineers on March 4, 1913.

NATHANIEL AUGUSTINE THAYER, Assoc. M. Am. Soc. C. E.*

Died September 10, 1925.

Nathaniel Augustine Thayer was born in South Boston, Mass., on November 4, 1880. He was the son of the late Edmond G. Thayer, a Civil Engineer and Architect of Boston, and Florence Hamilton Thayer.

Mr. Thayer was graduated from the Quincy, Mass., High School, and from the Lawrence Scientific School of Harvard University, in 1902, with the degree of Bachelor of Science. For several years after his graduation from college he was a Rodman and, later, an Instrumentman with the Cleveland, Chicago, Cincinnati, and St. Louis Railroad Company, after which, he spent a year as Structural Draftsman for J. R. Worcester and Company, Consulting Engineers, Boston, engaged in detailing and designing steel and reinforced concrete structures.

* Memoir prepared by Theodore Reed Kendall, Assoc. M. Am. Soc. C. E.

From 1906 to 1909, he served in the Bridge Engineering Department of the New York Central and Hudson River Railroad Company; as Structural Draftsman for Vielé, Blackwell and Buck, New York, N. Y., on the design of a wooden flume and steel and wooden head-gate for water-power development at Cobalt, Ont., Canada; and as Structural Detailer on buildings for Post and McCord, and, later, for Gunvald Aus, M. Am. Soc. C. E. In 1909 and 1910, he was employed as a Structural Steel Draftsman with the New York Public Service Commission, First District, on designs for subways and elevated railways for New York City.

During the latter part of 1910 and throughout most of 1911, Mr. Thayer was engaged by the Erie Railroad Company to work on plans for the wooden dock and for a grade-crossing elimination at Youngstown, Ohio, following which, he served as Structural Steel Designer for the Board of Education, School Building Department, New York City, and also held the position of Assistant Engineer with the Board of Transportation of the City of New York, of which, Robert Ridgway, President, Am. Soc. C. E., is Chief Engineer.

Through intensive study Mr. Thayer became a Certified Public Accountant in the State of New York. He undertook this work in order to be thoroughly grounded for an independent study which he was making of industrial corporation finance.

He died of pneumonia on September 10, 1925, at St. Luke's Hospital, New York City, after an illness of only four days, and is survived by his mother, Mrs. Edmond G. Thayer, of Wellesley Hills, Mass., a sister, Mrs. Raymond E. Huntington, and two brothers, Frederick G. Thayer, of Wellesley Hills, and Lucien H. Thayer, of Dorchester, Mass.

He was a member of The Harvard Engineering Society, and the Unitarian Church.

Mr. Thayer was elected a Junior of the American Society of Civil Engineers on April 5, 1910, and an Associate Member on April 2, 1912.

WILLIAM OLIN WALKER, Assoc. M. Am. Soc. C. E.*

Died May 8, 1925.

William Olin Walker was born at Chuckey City, Tenn., on July 18, 1890. His father, Thomas S. Walker, and his mother, Mary (Dooley) Walker, were descendants of the sturdy early settlers of Tennessee and Virginia.

Mr. Walker was educated at the Morristown, Tenn., High School, De Pauw University, Greencastle, Ind., where he took a Liberal Arts Course, and in the Extension Division of the University of Wisconsin. He was not graduated from De Pauw University, although for several years he alternated periods of college work there with periods of engineering employment for which he early showed an inclination.

During the summer of 1907, he was Rodman on the Cincinnati, New Orleans and Texas Pacific Railway Company (Southern Railway System),

* Memoir prepared from information on file at the Headquarters of the Society.

at Oakdale, Tenn., and during the summer of 1908, was employed in a similar capacity with the County Highway Engineer at Greenville, Tenn. From January to September, 1909, he served as Rodman with the Southern Railway System, and as Rodman and Instrumentman with the County Highway Engineer at Morristown, Tenn. During the summer of 1911, and part of 1912, he was Masonry Inspector with the Louisville and Nashville Railroad Company, resigning to accept a position as Building Inspector with the Illinois Central Railroad Company on construction at Paducah, Ky., and other locations.

After another year at De Pauw University (1913-14), Mr. Walker re-entered the service of the Illinois Central Railroad Company, and attained a varied and broad experience in railroad engineering—maintenance of way work, location, construction, and office practice—as follows: From October, 1915, to February, 1918, he was Instrumentman on maintenance of way, and had charge of laying out work on the construction of a new yard at Baton Rouge, La., and terminal facilities at Mattoon, Ill. From February to August, 1918, he had charge of a field party on a 100-mile preliminary survey for a new trunk line. In August, 1918, he became Acting Resident Engineer at Champaign, Ill., in charge of the construction of a classification yard. From January, 1919, to January, 1920, he was employed as Engineer Accountant at various points on the Illinois Central System, reclassifying accounts to prorate the cost of construction and improvements between the Railroad Administration and the Railroad Corporation.

In January, 1920, Mr. Walker entered the employ of the Sinclair Refining Company, at Coffeyville, Kans., as Cost Engineer. His work consisted of keeping detailed costs on the construction of a $5 000 000 refinery, also of making appraisal of an old refinery plant. His ability and thorough, painstaking work won recognition and he was transferred to the general office of that Company at Chicago, Ill., in April, 1921. In this position he checked cost reports from all the Company's refineries, and also assisted in reclassifying accounts for insurance. In December, 1921, he was given charge of the cost work in connection with the construction of new refineries at Coffeyville, Kansas City, Mo., Houston, Tex., and Marcus Hook, Pa., with headquarters at Coffeyville. During October, 1923, he was transferred to the refinery at East Chicago, Ind., as Assistant Engineer of Construction, in addition to his duties of Cost Engineer at the East Chicago Plant. The completion of the construction of the new refinery at East Chicago, in November, 1924, marked the approximate completion of the $40 000 000 construction program of the Sinclair Refining Company, in which Mr. Walker took an active part from its beginning.

On the completion of this work he accepted a position as Engineer for the H. W. Nelson Construction Company of Kentucky, with headquarters at Fulton, Ky., where he had charge of the construction of approximately thirty miles of the new line of railroad being built by the Illinois Central Railroad Company, from Edgewood, Ill., to Fulton.

A Convention of the Association of Engineers of the Mid-South was held at Memphis, Tenn., on May 8, 1925. Many prominent engineers were brought together at this meeting, especially men of the profession from the Central

Southern States, the majority of whom were members of the Society. The United States Government Engineers extended the Association an invitation to visit the Government revetment plant about twelve miles south of Memphis, near Coahoma Bend. Soon after leaving the revetment plant on their return to the city, one of the boats, the United States Steamboat *Norman,* sank in the Mississippi River. Mr. Walker and twenty-two others lost their lives in this disaster.

Perhaps two of Mr. Walker's most striking characteristics were his enthusiasm for his work and his keen interest in civic affairs. He was highly esteemed by his employers, respected and admired by his associates and subordinates, and known by his friends to have had fine ideals and deep convictions. He was a man of strong character, who never flinched from his duty, and yet was possessed of a tender devotion to his family and an innate courtesy and kindliness toward all.

Early in life he became a member of the Methodist Episcopal Church, of which he remained a faithful and consistent adherent. As a Knight Templar, Mr. Walker was active in Masonic circles.

In 1922, he was married to Mattie Sue Berry, of Barnesville, Ga., who, with two children, Meriam and Edith, survives him. He is also survived by four sisters, Mrs. E. T. Hall, of Brookfield, Mo., Mrs. R. B. Walker, of Walla Walla, Wash., and the Misses Clara and Eunice Walker, of Morristown, Tenn., and one brother, F. C. Walker, of Chicago, Ill.

Mr. Walker was elected a Junior of the American Society of Civil Engineers on March 7, 1921, and an Associate Member on May 28, 1923.

GEORGE BURNHAM, JR., Affiliate, Am. Soc. C. E.*

DIED NOVEMBER 22, 1924.

George Burnham, Jr., the son of George and Anna (Hemple) Burnham, was born in Philadelphia, Pa., on November 30, 1849. His father was associated with Matthias Baldwin in the early days of locomotive building and, later, was Senior Member of the firm of Burnham, Parry, and Williams, predecessor of the Baldwin Locomotive Works.

George Burnham, Jr., was educated in the public and private schools of Philadelphia, preparatory to entering the Rensselaer Polytechnic Institute, at Troy, N. Y., from which he was graduated in 1872 with the degree of Civil Engineer.

During the summer vacation of 1871, he was employed as Rodman with a field party on railroad work for the Bennett's Branch Extension, connecting the Allegheny Valley Railroad with the Philadelphia and Erie Railroad.

After his graduation in 1872, Mr. Burnham was engaged on surveys in Fairmount Park, Philadelphia, and as Leveler on a railroad survey from Paoli to Chester, Pa., for the Pennsylvania Railroad Company, under the late James McCrea, M. Am. Soc. C. E., who afterward became President of the

* Memoir prepared by Edward B. Guthrie, M. Am. Soc. C. E.

Company. Samuel Rea, Hon. M. Am. Soc. C. E., was also a member of this party.

In 1873 and 1874, Mr. Burnham was Rodman on the location and construction of a branch of the Pennsylvania Railroad from Williamsburg, Pa., to the Springfield, Pa., iron ore mines, and in 1874 and 1875 served as Leveler on the Bridge Division of the Bound Brook Railroad, Philadelphia to New York, N. Y. (now the Reading Central).

With the late D. McN. Stauffer, M. Am. Soc. C. E., he had entire charge of the construction of the bridge over the Delaware River which, with trestle approaches and bridges over canals on the Pennsylvania and New Jersey sides, was more than a mile in length. During this work he was stationed at Yardleyville, Pa.

On the completion of the bridge, Mr. Burnham returned to Philadelphia and, after 1875, when he entered into business with Mr. Nelson Stowe in the manufacture of the Stowe flexible shaft, never again engaged in the practice of his profession. From 1875 to 1880, he was Treasurer of the Barr Pumping Engine Company and Managing Director of the Spiral Weld Tube Company.

On September 1, 1880, he became associated with the firm of Burnham, Parry and Williams and served in the Treasurer's Department until January 1, 1896, when he became a member of the firm. On December 31, 1906, he retired from active business. His other business connections during these periods of active work and after retirement were, as follows: Director of the Central National Bank; Keystone Telephone Company; Keystone Coal and Iron Company; C. S. Wheeler Manufacturing Company; United Utilities and Service Company; Mineral Development Company; and the Integrity Trust Company.

Mr. Burnham was deeply interested in civic affairs, both municipal and Federal, and for many years was a leader in movements for the betterment of Philadelphia, in the words of one of his associates, "not asking for office, emolument, not even fame, only to be allowed to serve the city he loved." It was in this work and other civic matters that Mr. Burnham was truly notable, rather than in his career as an engineer which covered a period of only four years.

He was connected with various civic organizations, as follows: In 1880, he became a member of the Citizens Municipal Association and was also a Charter Member of the Philadelphia Civil Service Reform Association of which he was President in 1904. In 1890, he became a Charter Member and for ten years was President of the Philadelphia Municipal League. In 1894, this organization with the City Club of New York issued a call for the foundation of the National Municipal League of which Mr. Burnham was Treasurer for twenty-five years. In 1892, he was appointed a member of the Committee of 100 in Philadelphia and took a prominent part in its activities. In 1904, he became a member of the Committee of 70 and was engaged in its work at the time of his death. In 1905, he became a Charter Member of the City Club of Philadelphia and served as its President for ten years. In 1913, he served as a member of the Committee of 100, the purpose of which was to forward the election of Mayor Blankenburg. In 1921, he was appointed by the City Club of Philadelphia as its representative on the Board of the Thomas Harrison

Municipal Trust. He was made Chairman of the Board and held this office until his death.

On January 15, 1924, a Memorial Meeting was held at the Social Service Building in Philadelphia, at which addresses were delivered on the character and activities of Mr. Burnham by the following co-workers in reform matters: Messrs. Thomas Raeburn White, whose subject was, "Services in Politics"; Samuel S. Fels, who spoke on "Municipal Research"; W. W. Montgomery, on "Civil Service Reform"; Sherman C. Kingsley, on "Philanthropy"; J. Henry Scattergood, on "The Man"; Richard S. Childs, on "Municipal Reform"; and James G. McDonald, on "The City in Foreign Affairs".

To depict the fine character of Mr. Burnham the following editorial is quoted from the *Philadelphia Public Ledger:*

"George Burnham

"By the death of George Burnham, the city has lost one of its best friends. He stood confidently for everything that was honorable and upright and law-abiding in municipal government. He devoted time and strength and liberally gave of his own means in the furtherance of causes in which he believed, not asking if they were popular or likely to be victorious. There was in the man a fine spirit of chivalry and loyalty that endeared him to all who strove beside him for better things. Modest to the point of diffidence, he never put himself forward, he never asked the credit for anything he did. He disliked to make speeches or even to preside at meetings, but he never shirked his duty in movements for the general welfare. In fact, there was no undertaking for the good of the city with which his name and his effectual effort were not identified. In these he was the friendly, kindly, devoted counselor and partner. He was an example of the political crusader who is not a fanatic: he was that rare phenomenon, a reformer with a sense of humor and a dispassionate willingness to hear the other side. Important factor that he was in business life, it is as civic patriot that Philadelphia will gratefully remember him; and his city will commend him to posterity as a shining pattern of high-minded citizenship."

To these tributes, the writer wishes to add his own, speaking of Mr. Burnham as a friend and acquaintance of many years, dating from 1871. His gift of friendship equalled, or even surpassed, all others, and those whose privilege it was to know him feel that the world is made poorer by his death.

Mr. Burnham was married to Anna B. Lewis, on April 14, 1881, who, with two sons, Enoch Lewis Burnham and George Burnham 3d, and two daughters, Mrs. Arthur Peck, and Mrs. Albert DeSilver, survives him. He is also survived by two sisters, Miss Mary A. Burnham, and Mrs. Theodore J. Lewis.

Mr. Burnham was elected a Junior of the American Society of Civil Engineers on January 6, 1875, and an Affiliate on July 2, 1890.

GEORGE LEE ANDERSON, Jun. Am. Soc. C. E.*

Died May 8, 1925.

George Lee Anderson was born in Poland, Ind., on October 7, 1894. As his mother died when he was about two years of age, he was sent to live with his grandparents near Sunman, Ind.

* Memoir prepared by Garner W. Miller, Assoc. M. Am. Soc. C. E.

He was graduated from the district school near Sunman in 1908 and entered the High School at Greensburg, Ind., from which he was graduated in 1913. In September, 1913, he entered Purdue University, West Lafayette, Ind., taking the course in Civil Engineering. After two years at Purdue, he entered the employ of the Surveyor of Benton County, Indiana, as Draftsman and Computer.

In September, 1916, Mr. Anderson entered the United States Government service as a Surveyman, attached to the U. S. Engineer's Office at Memphis, Tenn. He remained in this position until 1918, when he entered the military service as a Private in the 472d Engineers, serving in the capacity of Levelman and Plane-Table Operator until his discharge in February, 1919.

In March, 1919, he returned to the U. S. Engineer's Office at Memphis as a Surveyman, remaining there until September of that year, when he returned to Purdue University to continue his course in engineering. During the summer of 1920, Mr. Anderson was employed as a Draftsman and Computer by Ayres and Miller, Engineers, of Memphis. He returned to Purdue University in September, 1920, where he completed his course and received the degree of Bachelor of Science in Civil Engineering, in June, 1921.

From July, 1921, until August, 1922, Mr. Anderson was employed in the U. S. Engineer's Office, at Memphis, as a Structural and Mechanical Draftsman. He then entered the employ of the firm of Ayres and Miller, as Office Engineer, retaining that position until his death.

Mr. Anderson was of a modest and retiring disposition and spent his spare time in reading both engineering and classical literature, possessing a remarkable memory of both. He was thorough and painstaking in every detail of his work, and, although only a young man, was an engineer of ability and possessed a high standard of integrity and professional conduct.

He was a member of the Engineers Club of Memphis, and was among those who lost their lives when the Steamer *Norman* sank in the Mississippi River on May 8, 1925. On board this steamer was a party of engineers who had been on an inspection trip down the river as part of the program of the Convention of the Association of Engineers of the Mid-South held in Memphis on May 7 and 8, 1925. He is survived by his father, Mr. Oliver C. Anderson, of Sunman, Ind.

Mr. Anderson was elected a Junior of the American Society of Civil Engineers on June 19, 1922.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

COMING MEETINGS

Board of Direction Meetings

January 18-19, 1926:

A Quarterly Meeting will be held at New York, N. Y.

Monthly Meetings

December 9, 1925:

8:00 P. M.—A regular business meeting of the Society will be held and a paper by Henry S. Prichard, M. Am. Soc. C. E., entitled "The Utilizable Capacity of Steel Members of Structures" will be presented for discussion. This paper is published in November, 1925, *Proceedings*.

Annual Meeting, New York, N.Y.

January 20, 21 and 22, 1926:

Seventy-third Annual Meeting of the Society.

January 20, 1926:

9:00 A. M.—Social Hour.

10:00 A. M.—Annual Meeting and Presentation of Medals and Prizes for Papers.

2:30 P. M.—Presentation and Discussion of Committee Reports.

7:30 P. M.—President's Reception and Dinner Dance.

January 21, 1926:

9:00 A. M.—Social Hour.

10:00 A.M. to 4:30 P.M.—Meetings of Technical Divisions.

8:00 P. M.—Entertainment and Smoker.

January 22, 1926:

All-Day Excursion.

The Reading Room of the Society is open from 9 A. M. to 6 P. M. every day, except Sundays, New Year's Day, Washington's Birthday, Memorial Day, Fourth of July, Labor Day, Thanksgiving Day, and Christmas Day; during July and August it is closed at 5 P. M.

Members, particularly those from out of town, are cordially invited to use this room on their visits to New York, to have their mail addressed there, and to utilize it as a place for meeting others. There is a file of 255 current periodicals, the latest technical books, and the room is well supplied with writing tables.

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